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THE CONSTRUCTION OF THE LAGUNA DAM, COLORADO RIVER, ARIZONA.

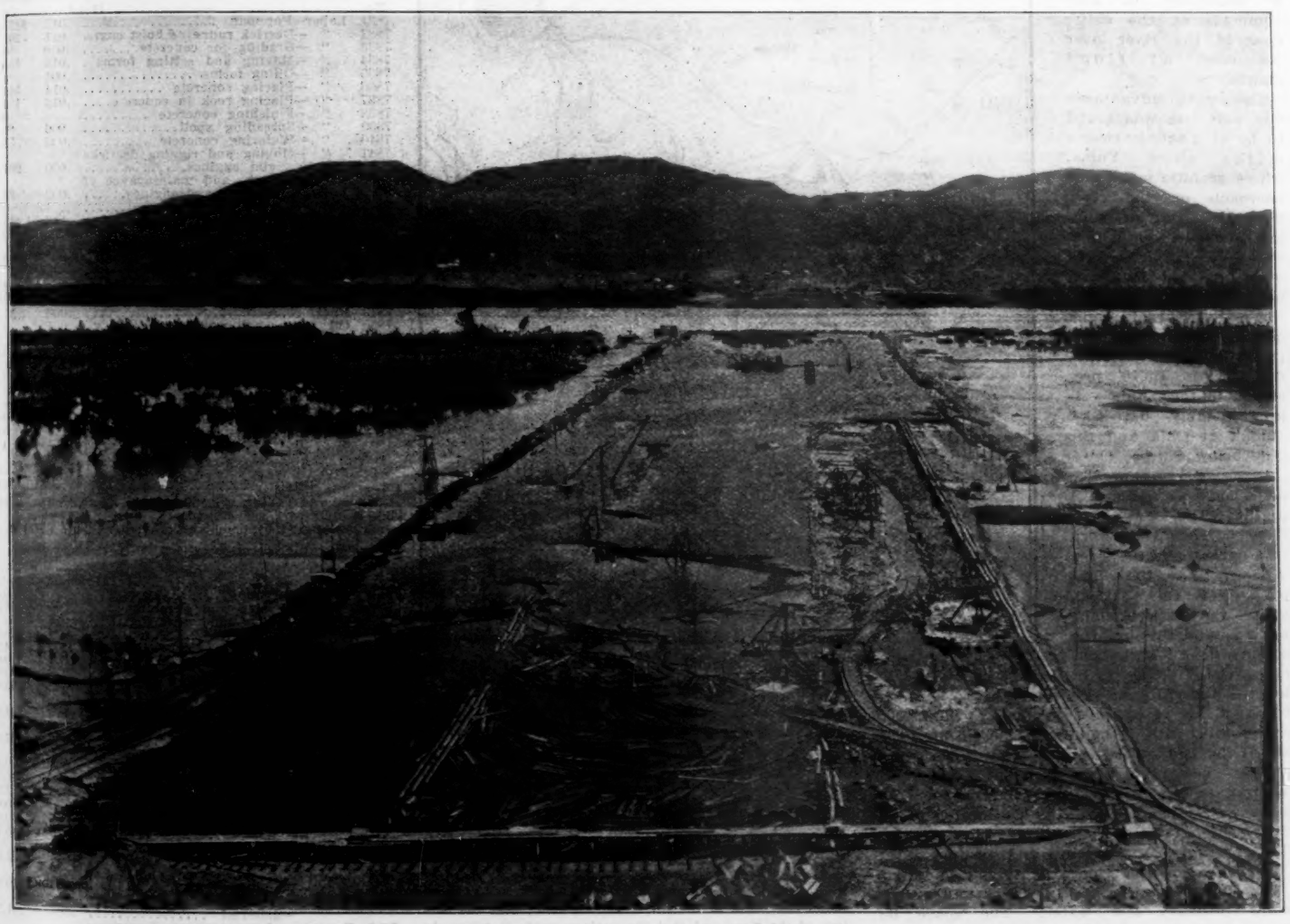
By EDWIN D. VINCENT.*

The Yuma project of the U. S. Reclamation Service, of which the Laguna Dam is the key, contemplates the irrigation of the valley lands of the Colorado and Gila rivers in the vicinity of Yuma, Ariz. Surveys made during the winter of 1903-4 showed that the area of irrigable land on both sides of the Colorado River was about 90,000 acres.

The Colorado River, aptly termed the "Nile of America," presented more than average difficulties to the engineers of the Reclamation Service in their plans to divert the water from its unstable channel to the adjacent lands. In the course of many centuries the river had built for itself a delta to the Gulf of California, consisting of sand and alluvial mud, whose depth in the Yuma territory the drills failed to establish. The river valley immediately above Yuma is from one to six miles wide. The river channel proper is from 1,000 to 4,000 ft. in width, and the slope is approximately 1 ft. per mile. The volume of discharge varies from about 4,000 sec.-ft. at extreme low water to over 100,000 sec.-ft. during flood seasons. At the latter time the stream overflows its banks, submerging the valley lands to a depth of from 1 to 6 ft. This inundation, due to the melting snows at the river sources, occurs annually, and generally lasts from May until August.

The stream, along its lower reaches, is continually cutting its banks and changing its channel. Old water courses demonstrate that as the river is to-day so it has been for ages past; its meanderings limited only by the rock bluffs bounding the valley through which it flows. By reason of this cutting of banks, the stream becomes a very heavy silt bearer after leaving the mountains, and an immense amount of sediment is continually carried in suspension or rolled along its bottom, at times amounting to over 1 part in 50, and averaging through the year .027 by weight.

The size and uncertainty of the river, the shifting channel and unstable banks, the yearly recurring inundations, variations in volume from low water to flood heights and the immense amount of silt carried by its yellow waters, made the problem of the control of the stream unique in the history of American irrigation. The surveys and borings demonstrated that a high dam to serve the lands about Yuma was impossible. Experiments made by private concerns in taking out water from the lower Colorado River, without considering the control



CONDITIONS AT THE LAGUNA DAM AT THE HEIGHT OF THE SUMMER FLOOD IN THE COLORADO RIVER.

*Resident Engineer, U. S. Reclamation Service, Yuma, Ariz.

of the stream itself, have proved to be extremely hazardous undertakings, menacing not only the enterprise itself by either tearing out headgates or by bar-forming processes, leaving such structures silted up and far inland, but also jeopardizing entire communities, as witnessed by the recent runaway river into the Imperial Valley and the Salton Sink.

With these requirements fully recognized, and with no bedrock for a base, the problem presented to the engineers of the Service was to build a structure founded on the sand and silt of the valley—one that would fully control the river and hold it within certain prescribed limits; added to this was the disposition of the silt, which is one of the most difficult features of the undertaking.

Nothing of the kind has ever before been undertaken in this country. It is true that in gaining control of the river at the Mexican intake to the Imperial Canal, a dam was thrown across the stream, forcing it back into its original channel to the sea, but this structure was simply a dike over which water was never intended to flow, while the Yuma project demands a weir capable, if necessary, of accommodating the entire flow of the river over its crest at flood stages.

The most advantageous weir site was found to be at Laguna, twelve miles above Yuma, where granitic mountains encroach on the river valley, leaving an opening about a mile wide. The type of weir selected was one that has been in successful operation for many years in India and Egypt, under practically identical conditions with those presented in the Yuma valley; notably on the Jumna at Okla and the Nile below Cairo.

Plans were approved, as shown in Figs. 1 and 2. [The views, Figs. 3 to 5, show the work in progress, and the large view, on the first page of this article, shows the work brought to a standstill by the summer flood.] Three concrete walls, 4,800 ft. in length and 57 and 93 ft. apart, are to extend from bluff to bluff. The crest wall, with a maximum height of 19 ft. above the bed of the stream, rests upon a row of 6-in. sheet piling from 12 to 20 ft. in length, incorporated in its base to cut off seepage. The crest wall is at Elev. 151. The water of the river will be raised about 10 ft., backing it up the stream nearly ten miles and forming a settling basin covering approximately 8 sq. mi.

The lower, or apron wall, is at Elev. 138, 3 ft. below ordinary low water. The space between walls is to be filled with broken stone, and an apron composed of derrick-size rock extends 40 ft. beyond the lower wall. The structure between walls will be capped with a concrete pavement 18 ins. in thickness. The slope over the weir from crest to apron wall is 1 on 12.

On the up-stream side of the weir a talus of broken rock, with an incline of 2 ft. horizontal to 1 ft. vertical, protects the concrete structure. This will be reinforced by deposited silt. In speaking of such a deposit, Mr. Burton Buckley, Superintending Engineer of the Indian Public Works Department, says:

> In cases where dangerous parallel currents are not likely to occur, a 6-in. deposit of silty mud in the channel of a river above a weir is a better security from underscour than foundations 50 ft. deep.

The dam when completed will be 4,800 ft. long between abutments, 19 ft. high in the river channel and 226 ft. in width up and down stream.

For description of the sluiceways, gates and intake canals [see Fig. 2] the Third Annual Report of the Reclamation Service is quoted:

> At the east end of the weir, constructed in solid granite rock and excavated to the depth of low water in the river, will be a sluiceway 116 ft. wide. At the west end the sluiceway, also in granite, will be only 40 ft. wide, but will be planned so as to permit enlargement to meet possible future needs. These sluiceways will be closed by large gates operated by hydraulic machinery. The diversion canals will take their water above these gates from the sides of the sluiceways. The area of these sluiceways being so great, the water movement toward the canal will be slow and most of the sediment will be deposited before reaching the canal intake. When this has accumulated to a considerable extent the sluicegates will be opened. It is estimated that the capacity of each sluiceway will be approximately 20,000 sec.-ft., and that this great volume of water will carry out with it the sediment deposited above the intake of the canal. The ordinary low-stage flow of the Colorado River is from 3,500 to 4,000 sec.-ft., so the capacity of the sluiceways will be about five times the low-water flow of the river. These figures are given for comparison only.
>
> As a result of a number of experiments it has been found that most of the silt is carried along near the bottom of the river and that the surface water is relatively free from sediment. It is planned, therefore, to take the water into the canals by a skimming process over a long row of gates, so that the canal can be filled by drawing but one foot in depth of water from the surface of the river. The main sluicegates will be of the Stoney type of iron roller gate. There will be three on the Arizona side and one on the California side. Each will be 36 ft. wide and 18 ft. high. If the settling capacity in the sluiceways is not sufficient for the exclusion of enough of the silt they may be extended up-stream so as to increase the length of time required for the water to pass through them, thus permitting the settlement of a greater proportion of the silt. . . .
>
> The capacity of these canals at their intakes will be 1,600 sec.-ft. on the Arizona side and 200 [illegible] on the California side. The amount of silt that will be delivered into the Arizona canal if diversion were made directly from the stream would approximate [illegible],000 cu. yds. of wet mud, by volume, and it is not believed to be possible for a canal to continuously operate successfully for the irrigation of lands along the valleys of the Colorado unless some very substantial arrangements are made at the headworks for handling this silt. This, and the fact that the water must be held to a [illegible] level at the canal heading for all stages of the river, is believed to justify the expenditure proposed for these headworks.

The approximate quantities for the weir and headworks are: rock excavation, [illegible],000 cu. yds.; earth excavation, 360,000 cu. yds.; rock fill, 350,000 cu. yds.; concrete paving, [illegible],000 cu. yds.; concrete core walls, 28,000 cu. yds.; concrete gate piers, guide walls, lining, etc., 7,000 cu. yds.; sheet piling, 90,000 lin. ft.

The problem presented to the construction engineers was to push a rock-filled dam 4,800 ft. long and 226 ft. wide, with three concrete core walls, across the valley and to build it practically under water, as three-fourths of the structure as planned is below the seepage line, while its highest portion is but 3 ft. above extreme high water, the apron and lower walls extending 10 ft. below ordinary low water.

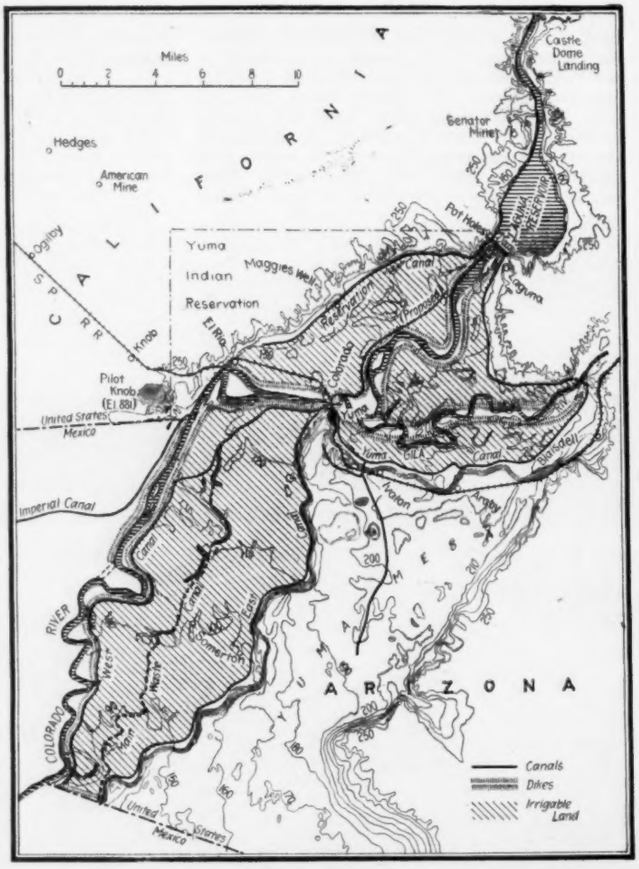


FIG. 1. MAP SHOWING THE LOCATION OF THE YUMA IRRIGATION PROJECT, U. S. RECLAMATION SERVICE, ARIZONA-CALIFORNIA.

TABLE OF COST DATA, LAGUNA DAM, ARIZONA SIDE.

Cost Account of Concrete Paving, July, 1907.

No.		Cost—Hundredths	Total.
7831	Labor—Foreman	.012	$125.00
7832	" —Derrick runners & hoist engrs.	.011	105.78
7833	" —Grading for concrete	.010	98.94
7834	" —Moving and setting forms	.013	133.42
7835	" —Oiling forms	.001	5.91
7836	" —Placing concrete	.014	141.73
7837	" —Placing rock in concrete	.013	134.82
7838	" —Finishing concrete	....	0.00
7839	" —Spreading spoil	.006	60.61
7840	" —Watering concrete	.006	58.22
7841	" —Moving and rigging derricks and engines	.020	201.45
7842	" —Repairs and maintenance of derricks and engines	.002	23.20
7843	" —Repairs to equipment	....	0.00
7844	Hauling concrete and rock	.143	1,445.11
7845	Concrete	.729	7,370.57
7846	Repairs and maintenance of derricks —lumber and timber	....	0.00
7847	Repairs and maintenance of derricks and engines—material	.000	4.79
7848	Repairs to equipment	....	0.00
7849	Fuel	.003	31.50
7850	Oil, grease and waste	.001	5.96
7851	Material and supplies	....	0.00
7852	Miscellaneous	....	0.00
7853	Light and water	.010	101.01
7854	Equipment	.002	19.15
7855	Depreciation of derricks & hoists	.003	32.11
7856	Depreciation of stationary engines	.001	8.78
7857	Depreciation of rails	....	0.00
7858	Pumping	....	0.00
		1.00	
	Total		$10,108.03

1,585 cu. yds. paving = 3,169 sq. yds., 18 ins. in thickness.
6.3773 unit cost per cu. yd.
Concrete mixed by machinery; average haul, 1,100 ft.

Cost of Excavation, Class 2, November, 1907.

		Cost—Hundredths	Total.
7661	Labor—Foreman	....	0.00
7662	" —Clearing and grubbing	.005	16.50
7663	" —Plowing and scraping	.017	50.12
7664	" —Teams hired	.024	70.09
7665	" —Teams U. S.	.001	4.33
7666	" —Hand work and sluicing	....	0.00
7667	" —Running dredge	.226	672.06
7668	" —Pumping	.032	95.83
7669	" —Repairs and maintenance of dredge plant	.146	434.66
7670	" —Repairs and maintenance of pump plant	.006	17.89
7671			
7672	" —Moving dredge	.004	12.81
7673	" —Moving pumps	.006	16.55
7674	" —Miscellaneous	.006	17.51
7675	Fuel for stationary boilers	....	0.00
7676	Fuel for dredge	.255	760.83
7677	Fuel for pumps	.046	135.77
7678	Lubricating oil, grease and waste on dredge	.021	63.41
7679	Lubricating oil, gease and waste on pumps	.004	10.26
7680	Repairs and maintenance of dredge —material	.067	201.25
7681	Repairs and maintenance of pumps—material	.008	24.71
7682			
7683	Material and supplies	.006	16.47
7684	Forage	.007	19.93
7685	Light	.008	24.30
7686	Miscellaneous	.000	1.29
7687	Equipment	.002	4.96
7688	Depreciation on dredge plant	.098	286.68
7689	Depreciation on pump plant	.007	21.86
		1.00	
	Material moved during November, 1907, 12,490 cu. yds., at $.238		$2,979.98

Note: The excavation for the dam is being made by suction dredge with distillate power, a 65-HP. gas marine engine being belted to an 8-in. centrifugal pump, with a 12-HP. gas engine running the cutter and swinging gear —lift, 18 ft. through 500 ft. of 11-in. riveted pipe.

The construction of the work was let under contract to Messrs. J. G. White & Co., of New York City, who began operations on July 20, 1905, and were to finish the structure in two years. The contractor began on both sides of the river at once, planning to have the weir meet in the center of the stream. In practical work, however, many conditions were encountered which had not been taken into consideration or could not have been foreseen when the bid was made. The work is located in the desert, twelve miles from the railroad. The months of June, July, August and September are extremely hot; the high-water season virtually stops construction for three months in the year, but during that time a large force must be maintained in order to prevent damage by the flood. The poor quality of the rock for the fill of the dam demands extensive sorting. The indifferent labor, consisting largely of Mexicans and Indians; the experimental character of the plant equipment; the inadequate shipping facilities from the railroad; these, with other causes, increased the expense far beyond the income and hindered progress. The contractors, after eighteen months of operation, with about 35% of the work done, were relieved and their plant taken over on Jan. 23, 1907, for construction by the Reclamation Service direct.

A considerable change in the working force was found necessary in order to accomplish better results. Storehouses were constructed to economically handle equipment and supplies. The Government found it necessary to take over the mercantile stores and boarding houses that had formerly been conducted by outside parties. The railway system was entirely remodeled and considerable of the old equipment was taken out and either remodeled or replaced by suitable machinery. The transportation question from the railway station to the dam is still a galling one, owing to the poor roads and uncertain river, but is expected to be soon remedied, as the Government is now building a railroad up from Yuma to this point.

The account number system was adapted to the needs of the work. Finely segregated and readily accessible cost data facilitate comparison of labor and material costs, and are quite necessary to build up a proficient organization and to accomplish certain results. A monthly summary of two items of construction, concrete paving and earth excavation, are given in the table on page 214, showing their subdivisions of cost for ready analysis.

The rock excavation in the sluiceways is being blasted down, loaded into cars and transported to points of consumption by industrial locomotives. The rock crushing and concrete mixing is done by machinery which, owing to the growing scarcity of wood fuel, has recently been changed from steam to distillate power, at a very considerable saving in cost.

The pumping of sumps is accomplished by gas engines, and in fact the Government is, wherever possible, gradually changing to that power, with distillate for fuel.

With a fairly good type of slide-valve engine, say 30-HP. size, 35 lbs. of steam at 100 lbs. pressure per HP. hr. is good work. To generate this 35 lbs. of steam would require 12½ lbs. of wood, which costs us delivered at the boilers $5.50 per ton, or $.00275 per lb., or $.03437 per HP.-hr., as compared with a No. 1 distillate consumption of ⅛-gal. per HP.-hr., as shown by our records. This distillate costs, delivered at the engines, 16 cts. per gal., or 2 cts. per HP.-hr. In one of the sumps where pumping was started on Sept. 27, 1907, 3,200 gals. of water have been handled per minute on a 16-ft. lift, and through 72 ft. of pipe. This work is being done with a centrifugal pump consuming 26 HP. For 1,536 hours this run was continuous and consumed 4,992 gals. of distillate at a cost of $799. Had steam been used, the fuel cost for this work would have been $1,373, showing a saving on the methods employed of $574.

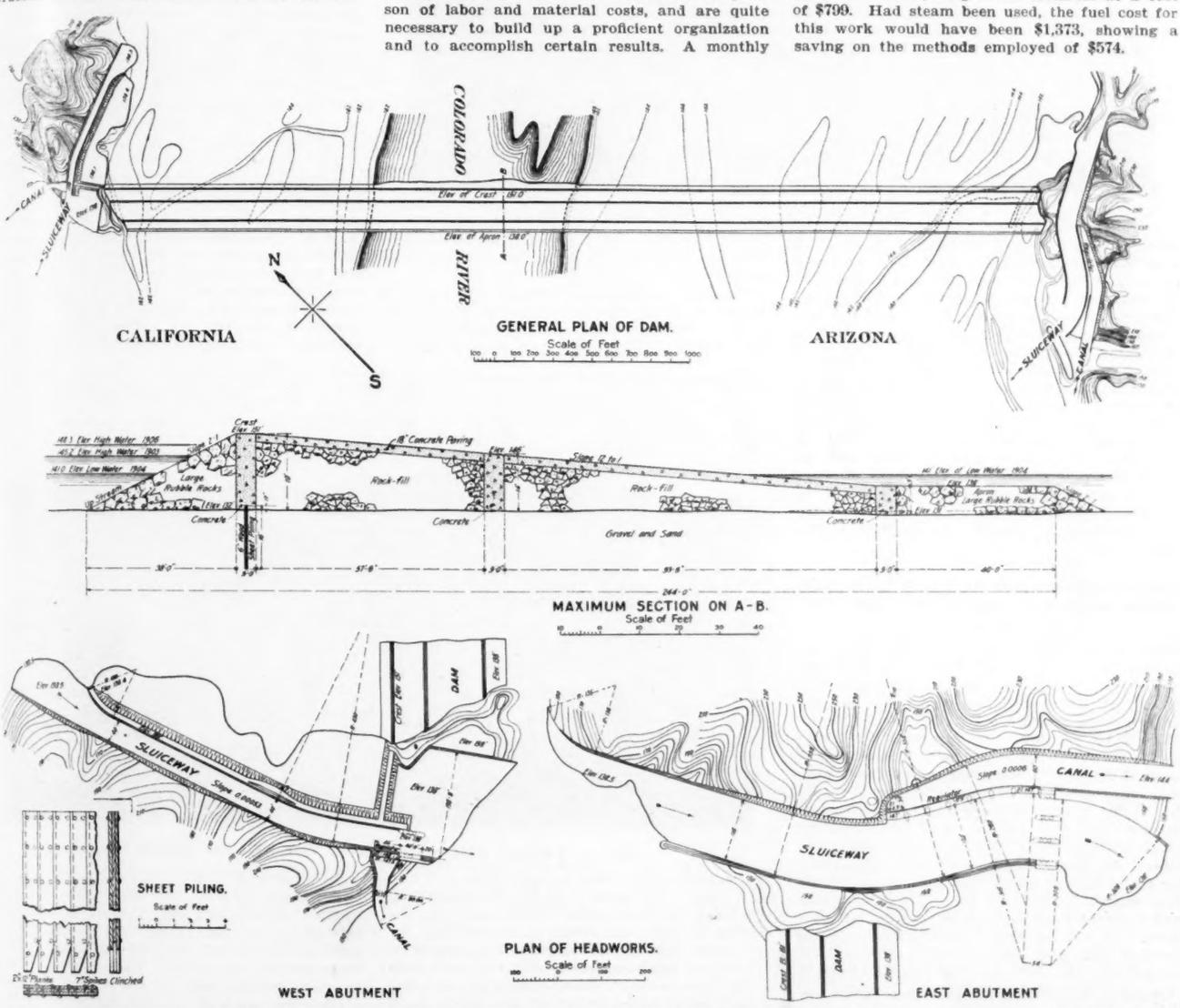


FIG. 2. GENERAL PLAN, MAXIMUM CROSS SECTION AND DETAILS OF LAGUNA DAM ACROSS THE COLORADO RIVER, ARIZONA-COLORADO.

About 25% of the construction work on the dam has been completed by the Government in the past ten months, making a total of 60% accomplished since the beginning of the work. During the greater part of June and July our main forces were required to protect the work against the onslaught of the swollen river. Willow mats, sand bags, and trainload after trainload of rock were opposed to the swift shore-currents on the California side during 24 hours of the day, and for a time the outcome of the contest was doubtful. The forces were finally drawn off, with the fight won, no damage having been accomplished by the river, with the exception of a large flood expense taxed to the work.

On August 8, 1907, on account of limited funds, the force was reduced to 170 men, or

FIG. 3. LAGUNA DAM FROM ABOVE THE CALIFORNIA SLUICEWAY.

about one-third its former strength, since which time it has not been augmented. The work now is confined to the Arizona side, and to one shift of eight hours duration, except on the dredge, which operates through the 24 hours of the day.

One of the sluicegate piers in the Arizona sluiceway has been constructed. The California end of the dam, with the exception of paving, has been extended to within 250 ft. of the river channel. The dam on the Arizona side is now being built into the river, and when the present cofferdam across the front is reached, the distance between ends will be about 800 ft. Thereupon the 800 ft. of river section will be cofferdammed and the ends of the weir joined, while the low water flow of the river is diverted through one or both of the sluiceways.

ELECTRICALLY OPERATED SLUICE GATES AND DROP-TIMBER REGULATOR GATES FOR THE LAGUNA DAM.

By F. W. HANNA.*

One of the most interesting engineering features of the present work of the U. S. Reclamation Service is the installation of the large sluice gates and their operating mechanism for the Laguna Dam, Yuma Project, California-Arizona. These gates are among the largest of their kind in the world, and are otherwise interesting for unusual details of design and operating mechanism. The designing of the gates was done by Mr. F. Teichmann, Engineer, U. S. Reclamation Service, and that of the electrical operating mechanism by Mr. O. H. Ensign, Consulting Engineer, U. S. Reclamation Service. The gates for the Arizona sluiceway, at the left end of the dam, were furnished and shipped to the dam site under contract by J. G. White & Co., of New York, and will be erected by Government force account. Advertisement is soon to be issued for the gates for the California sluiceway at the right end of the dam. [A description of the Laguna Dam and sluiceways is printed on the preceding pages.—Ed.]

The sluice gates for the Arizona sluiceway will be located below the crest of the dam, near the outlet of the sluiceway and immediately below the Arizona Canal intake. (See Figs. 1 and 4.)

*Engineer, U. S. Reclamation Service, Washington, D. C.

FIG. 4. LAGUNA DAM FROM ABOVE THE ARIZONA SLUICE

Two concrete piers (Fig. 1), each 8 ft. wide and 48 ft. long, will serve to divide the channel at this point into three equal widths, and with the abutments will form supports and settings for the gates and appurtenances. The height of these piers and abutments from the gate seats to the tops on which the operating car and machinery are to be supported will be 43¾ ft. Each of the three openings will have a clear waterway width of 33⅓ ft. and there are, therefore, required three service gates to control the outflow of this sluiceway. At a distance of 8 ft. above the service gate seats there will be another set of gate seats for emergency purposes. There is but one emergency gate provided for the three openings at these seats, it being the intention to transfer this gate from one of the openings to another as necessity demands. To facilitate this transfer openings will be left in the piers, opposite the emergency gate seats, to make a passageway for the gate. There will be a girder bridge without a floor over the tops of the piers and abutments above the position of the emergency gate seats. This bridge will be provided with a track for a car on which the operating machinery for the emergency gate will be located and which will be used in transferring this gate from one set of gate seats to another (see Fig. 2). A similar bridge, with a floor, will be placed above the service gate seats for the operating machinery for the service gates. Both bridges on the California side will be of this latter type, as the emergency gate will not have to be transferred from one position to another.

The four gates required for the Arizona sluiceway and the two required for the California sluiceway are each exactly alike in all respects. The bodies of the gates (Fig. 3) are constructed throughout of suitable plates and angles, with riveted connections, and are 17 ft. 11¼ ins. high and 34 ft. 9½ ins. wide. The thickness of the plates and the sizes of the angles throughout are quite accurately proportioned to the stresses brought upon them. The water pressure is to be taken directly by a skin plate on the up-stream face of the gate and transferred laterally to the piers at the side of the gate by means of four horizontal riveted girders. Each of these girders has a maximum depth at the center of 42 ins., a minimum depth at the ends of 27 ins. and is composed of flange angles and a web plate, reinforced with angle stiffeners at intervals of about 2½ ft. The three lower girders are equally spaced at a distance of about 5 ft., while the distance between the two upper ones is about 7½ ft. The four girders are interbraced by means of five cross girders, likewise composed of angles and plates. One of these cross girders is placed at each side of the gate and one in the center, the remaining two being placed, one on each side of the center at a distance of about 7½ ft. from it. For the purpose of preventing lateral motion of the gates there are placed on the sides of each gate, at the four corners, cast-iron rollers having an approximate length and diameter of 6 ins., which will have rolling contact with the gate frames. On the backs of the gates, at the ends of the girders, are attached cast steel strips, having finished surfaces 6½ ins. wide for transferring the water pressure through the roller

FIG. 5. END VIEW OF LAGUNA DAM UNDER CONSTRUCTION; ARIZONA SIDE.

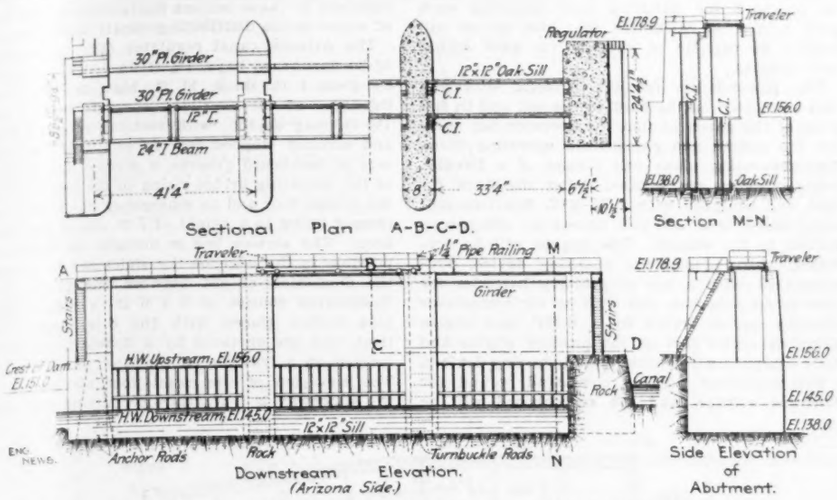


FIG. 1. PIERS AND ABUTMENTS FOR SLUICE GATES, LAGUNA DAM, ARIZONA END.

trains to the roller tracks on the gate frames. The lower edges of the skin plate and end web plates, reinforced with the leg of an angle and a narrow cover plate, are finished to a true surface to form a water-tight seat on the gate sill. Attached to the front of each gate, by means of angles, adjacent to the piers and abutments will be 3 × 3-in. needles of Oregon fir. These strips of wood will be so placed as to form snug contact with an exterior flange of the gate frame castings, and thereby prevent seepage around the ends of the gates.

The gate sills will be composed of wrought iron plates, fastened with lag screws to 12 × 12-in. white oak timbers anchored to the rock floor of the sluiceways. These timbers will be sunk into the rock to such a depth as to bring the gate seats on a level with the sluiceway floor and will be grouted, in position, with cement. The gate recesses in the piers and abutments for the reception and bearings of the sides of the gates will be 16 ins. deep and 37¾ ins. wide, and are to be lined with heavy ribbed iron castings, resting on a bed plate of the same material. The ribs of the castings will be external to the recesses and will be embedded in the surrounding concrete. The castings of the recesses of laterally adjacent gates will be bound together through the piers by means of turnbuckle rods, and those at the abutments are to be firmly attached thereto with anchor rods. The casting at the back of a recess and that on the down-stream face thereof, each have a vertical strip about 6 ins. wide, slightly elevated above the general casting surface, which surface is finished to facilitate the passage of rollers over them. Each roller train contains 26 cast-iron rollers, 6 ins. in diameter and 5 11-16 ins. in length, fitted between two ⅝ × 6-in. wrought iron plates, 19 ft. 10 ins. long. The rollers have bronze bushings and revolve about 1¼-in. steel pins that pass through both the bronze bushings and the wrought iron plates. The pins are shouldered and provided with threads and nuts at the ends, thus firmly gripping the sides of the roller racks together. The ends of the roller racks contain pulleys around which ½-in. crane chains pass. One end of each of these chains will pass over a sheave located on the operating bridge, thence to an attachment on a counterweight box. The other ends of the chains will be secured to small cast iron drums, also located on the operating bridges. By means of this arrangement, it will be possible to raise the roller trains to the full height of the position of the gates by hand operation of the cast iron drums referred to. There will also be attached to the top corners of each gate 1½-in. crane chains, which pass over pocketed sheaves, each pinned to the same shaft as the corresponding similar roller train sheave, thence to attachments on counterweight boxes.

The crane chain sheaves are mounted on a common axis, with a cast steel spur gear meshing with a bronze pinion. The bronze pinions corresponding to each side of a gate are keyed onto a cold-rolled steel shaft, extending from one set of sheaves above one side of the gate to those above the other. At the center of this shaft is mounted a worm wheel, engaging a worm

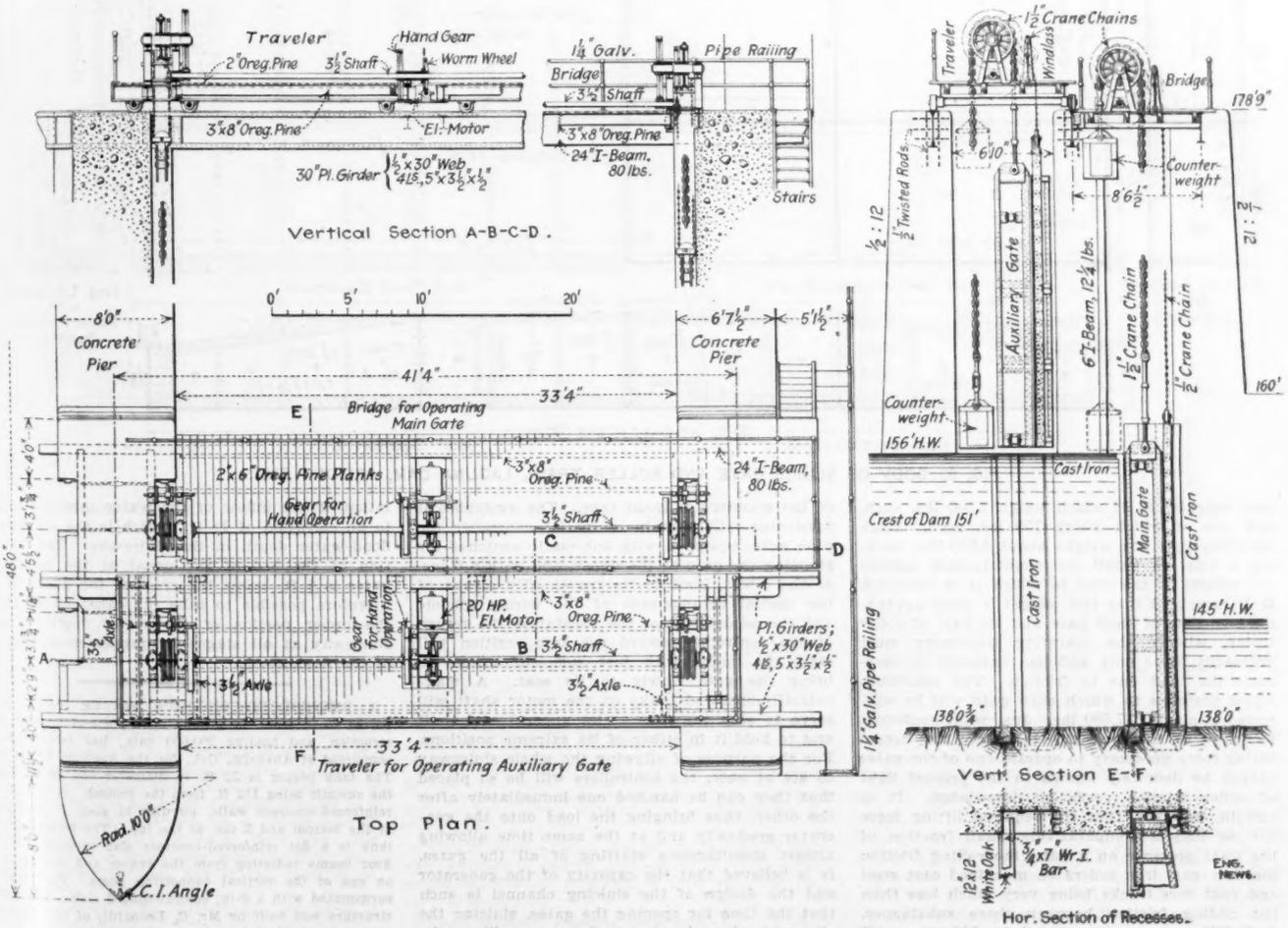


FIG. 2. SLUICE GATE, WITH OPERATING MACHINERY, LAGUNA DAM, ARIZONA END.

on a shaft coupled to the motor shaft running at right angles to the worm wheel shaft. The worm will revolve in a box of lubricating oil and the end thrust on its shaft will be taken by a thrust ball bearing. The electric motor for operating each service gate will be located on the operating bridge directly above the center of the gate, and that for handling the emergency gate will be similarly located on the traveler. There is also a means provided for raising each gate by hand with a ratchet and pinion connection through a spur gear to be mounted adjacent to the motor on the shaft carrying the worm wheel above referred to. The traveler is designed for hand propelling from one location to another by means of a ratchet and a pinion, meshing with a spur gear on an axle of the car.

The counterweight boxes are constructed of plates and angles, are 27 ins. square, out to out in cross-section, and are 34 ft. 9 ins. long. Each counterweight box will be loaded with 30 cast iron weights, 26 of which weigh 1,500 lbs. each, and two of which weigh 750 lbs. each. Each counterweight box weighs about 6,500 lbs., making a total of 48,500 lbs. counterpoised against the weight of the gate to which it is attached. It is computed that this weight is quite approximately that of each gate and its pair of roller trains, whence the operating machinery must ordinarily have only sufficient capacity to overcome the load due to friction. The maximum water pressure to which each gate will be subjected is about 337,500 lbs. Just what coefficient to apply to this quantity to obtain the actual lifting force necessary to operate one of the gates cannot be definitely known in the present light of roller friction coefficient knowledge. It is certain, however, that the required lifting force will be only a comparatively small fraction of the total pressure on a gate, the rolling friction for 6-in. cast iron rollers on machined cast steel and cast iron tracks being very much less than the sliding friction between these substances. A 20-HP. series motor, wound for 500 volts, will be provided for handling and operating each gate, and it is calculated that this motor will readily be capable of opening the gate within two minutes.

The power-house for the Arizona sluiceway will be located on the bluff to the left and in full view of the gates, so that the operator can oversee the motors and gates when operating them. The generating plant will consist of a 100-HP. triple-cylinder gas engine, using distillate for fuel, and equipped with a 75-KW. direct-current compound-wound 500-volt generator, direct-connected to the engine. The engine will be furnished with a magnetic sparking device, supplemented with a set of primary batteries for emergency purposes, and with an air compressor starting device driven by a 6-HP. gas engine using the same fuel as the primary engine and fully equipped with sparking and starting devices.

The generator will be compound wound for constant voltage, and the switchboard will be of the standard railroad type. The switchboard apparatus will contain a special controller for each gate, equipped with automatic switches for stopping the gate at the upper, and within 6 ins. of the lower, limit of its travel. By means of the special arrangement of the wiring scheme and the adjustment of the resistance, the return of the controller toward the zero position will start the motor with just sufficient torque to bring the gate slowly to its seat. A magnetically-operated brake on the motor shaft will serve to stop the gate at its upper travel limit and to hold it in either of its extreme positions. For the purpose of allowing the whole sluiceway to act at once, the controllers will be so placed that they can be handled one immediately after the other, thus bringing the load onto the generator gradually and at the same time allowing almost simultaneous starting of all the gates. It is believed that the capacity of the generator and the design of the sluicing channel is such that the time for opening the gates, sluicing the channel and again closing the gates will not be sufficient to cause serious fluctuations in the flow of water in the distributing canals of the project.

The Arizona canal regulator (Fig. 4) consists of 34 flashboard bays, 7½ ft. in width, separated by piers 1 ft. thick, 11 ft. high, including the thickness of the overlying concrete bridge, and 7½ ft. long on top, with vertical, rounded front and suitably stepped rear faces. There are two sets of flashboard grooves, a service line in front of the operating bridge rising to the full height of the bridge floor and an emergency line at the rear thereof rising to a height of 7 ft. above the canal floor. The service line is brought as closely as possible to the edge of the sluiceway to prevent the deposition of silt between the piers. The flashboards consist of 4 × 6 in. × 8-ft. Oregon pine timber placed with the 6-in. depth vertical, and are operated by a double hand winch located on a traveler on the floor of the bridge. The elevation of the canal floor above that of the sluiceway is 9 ft., thus permitting the regulation of the inflow of the water into the canal to an elevation of 18 ft., which is the maximum flood-water depth in the sluiceway. The elevation of the bed of the canal at the regulator gates is 4 ft. below the crest of the dam. It is therefore possible to take the canal water from the upper portion of the water depth in the sluiceway at all stages by proper manipulation of the flashboards.

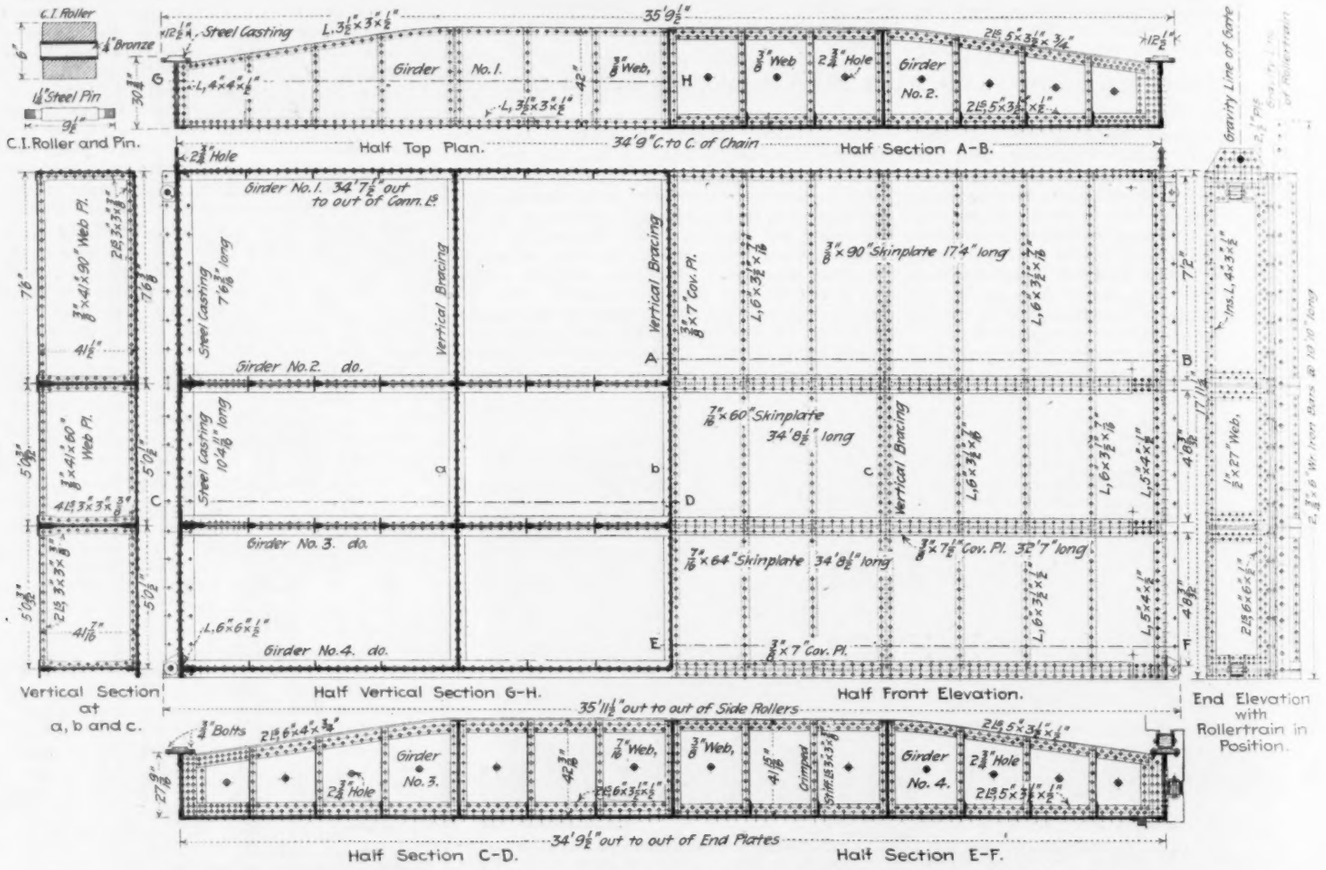


FIG. 3. BODY OF SLUICE GATE AND ROLLER TRAIN, LAGUNA DAM, ARIZONA END.

A REINFORCED-CONCRETE WATER TANK supported 70 ft. from the ground upon posts of similar construction, and holding 200,000 gals., has recently been completed at Anaheim, Cal., for the Anaheim Water Co. The tank proper is 32 ft. in diameter and 38 ft. high, the summit being 112 ft. from the ground. It is built of reinforced-concrete walls, circular in plan, 5 ins. thick at the bottom and 3 ins. at the top. The bottom of the tank is a flat reinforced-concrete slab supported on 12 floor beams radiating from the center and resting each on one of the vertical supporting posts. The tank is surmounted with a 2-in. conical-shaped slab. The whole structure was built by Mr. F. Leonardt, of Los Angeles, at a cost of $11,400.

THE RUN-OFF FROM SEWERED AREAS.*

The committee appointed by the Sanitary Section of the Boston Society of Civil Engineers to collect such data as may be in existence and to procure additional data concerning the relation between the rainfall upon and the run-off from sewered areas has succeeded in establishing a few stations for observations of rainfall and run-off, has studied the information thus obtained and formulated a few suggestions as to methods of procedure which may be of service to those interested in similar work, and is taking this opportunity to send out the following to parties who may be willing to cooperate with the committee.

For the purposes of this investigation, the committee is of the opinion that the apparatus mentioned below, if properly installed and cared for, will give satisfactory results.

INSTALLATION OF APPARATUS.

(1) AUTOMATIC RECORDING RAIN GAGE.—This should be located as near the center of the drainage area as practicable, at a point free from obstructions, such as buildings, trees, etc., preferably on level ground. On large areas the value of the results will be increased if more than one rain gage is installed.

(2) STANDARD RAIN GAGES.—These should be so placed as to show the variation of the precipitation over the drainage area under consideration. One gage should occasionally be used for the purpose of checking the automatic rain gage.

(3) AUTOMATIC RUN-OFF GAGE.—This should be installed preferably in a side chamber connected by a pipe with the drain to be observed, but where this is not practicable it may be installed in the drain itself, the float being protected by a surrounding chamber so arranged as to counteract the impact of the water. The clockwork and recording device may be installed in a manhole, but will give much better results if placed in a box above ground, so constructed as to exclude most of the moisture coming from the drain. Such boxes, like police boxes, have in some cases been placed on posts located either on the curb line of the street or on private property.

(4) MAXIMUM RUN-OFF GAGES.—At least three of these gages should be provided, one placed as near the automatic run-off gage as practicable, another below and the third above the automatic gage on the same line of drain, these gages to be used to check the automatic gage and to determine the slope of the water surface.

COST OF APPARATUS.

If ten stations are established calling for new apparatus, the following club prices may be obtained:

1	Recording rain gage	$100
2	standard rain gages at $10	20
1	run-off gage	45
3	maximum run-off gages	3
		$168

RECORDS.

There has also been considered the matter of a standard form of chart to be furnished by the committee, upon which the results obtained at the various stations are to be recorded, and sent in. A tentative form for the same has been prepared, and copies may be obtained upon application to the Secretary of Committee.

The committee deems it necessary that each observer should furnish, in addition to the charts, the data suggested in the following schedule:

(1) DRAINAGE AREA.—Map of drainage area, preferably to a scale of 100 ft. to 1 in. or multiples thereof, with elevation at street intersections noted, and tributary area computed.

Show all drains with sizes, invert elevations at manholes and rates of slope, catch basins and overflows.
All ponds, brooks and other natural or artificial channels.
Different areas making up drainage area colored as follows:
Swamp, dark green. Areas computed.
Wooded, light green with trees. Areas computed.
Meadow, light green with edges of high land brought out. Areas computed.
Streets, whether gravel, macadamized or paved and form of paving used. Areas computed.
State number of houses and other buildings with connections, and indicate manufacturing establishments, if any.

(2) RAIN GAGE.—Sketch showing location of rain gage with surrounding objects, approximate elevation above ground and distance from nearest trees, buildings, etc. Scale 10 ft. = 1 in.

Type of automatic recording gages used.
Sample dial or sheet.

(3) MEASURING STATION.—Plan and elevation of run-off measuring station, connection with recording gage, etc. 10 ft. = 1 in. or larger.

Type of automatic recording float gage used.
Sample dial or sheet.

The committee will be pleased to receive any suggestions or advice, and it is particularly desirous of cooperation on the part of those interested by the establishing of as many gaging stations as possible.

The committee will be glad to furnish, so far as lies within its power, any further information or suggestions concerning the scope of the work or the apparatus required.

It is intended that proper credit shall be given to those furnishing data, for their respective contributions. The committee expects to keep in close touch with those making observations, and give to each the benefit of the experience of other observers, and also the results obtained as soon as practicable.

While the Sanitary Section, through its Committee, has taken the initiative in starting this investigation the function of the committee will be merely that of a Clearing House. The value of the results must depend upon the number and interest of those who cooperate.

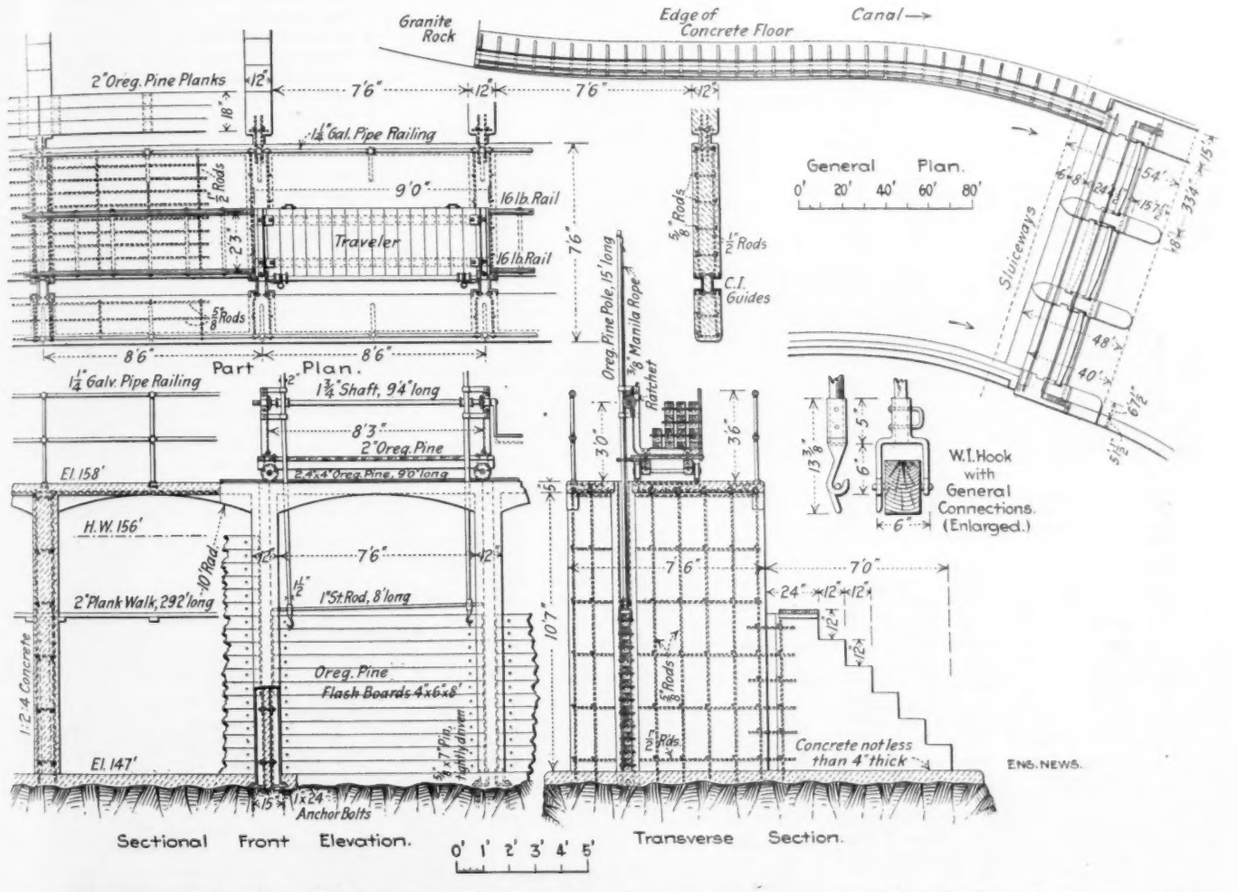


FIG. 4. CANAL REGULATOR DROP-TIMBER GATES, ARIZONA END OF LAGUNA DAM, WITH GENERAL PLAN OF CANAL REGULATOR AND OF SLUICEWAYS.

*Preliminary report of the Committee on Run-Off from Sewered Areas, Sanitary Section, Boston Society of Civil Engineers, dated Boston, Mass., Jan. 1, 1908.

(Signed) Irving T. Farnham, Chairman, George A. Carpenter, Hector J. Hughes, Harrison P. Eddy, L. M. Hastings, Arthur T. Safford, W. S. Johnson, Committee.

Address correspondence to the Secretary of the Committee, Harrison P. Eddy, 14 Beacon St., Boston.

REFERENCES TO LITERATURE UPON RELATION OF RUN-OFF TO RAINFALL IN SEWER DISTRICTS.

What should be the Capacity of Municipal Sewers in order to Carry off Safely the Water of Heavy Downpours? (Welche Maximal-Wassermengen haben stadische-Abzugs-Kanale während heftiger Gewitterregen thatsächlich abzufuhren?).—By Von C. Mank. Deutsche Bauzeitung, 18 Jahrgang, pp. 90, 128,286. (1874). (A continued article, with illustrations and tables by the chief engineer in Dresden).

Report on Sewerage Systems of Europe to National Board of Health.—By Rudolph Hering. (Pamphlet, about 1880).

Notes on City Sewers. (Note sur les Egouts des Villes).—Par Daniel E. Mayer. Annales des Ponts et Chaussées, Memoires et Documents, Tome 15 (1881), No. 5. Estimated that the sewers of Paris should be able to discharge per second a volume of rainwater equal to a third of the maximum rainfall per second $= \frac{0.125}{3}$ $\frac{1.77 \text{ ins.}}{3}$ cu. m. per hectare, or per hr.

Determination of the Size of Sewers.—By Robert E. McMath. Trans. Am. Soc. C. E., Vol. 16, p. 179. (April, 1887). (Size of a storm-water sewer under the conditions that it shall carry off the water of the great storms of the locality, and that no excess of size and cost be allowed; based on experience with St. Louis sewers).

Proportions of Rainfall Reaching the Sewers.—By Emil Kuichling, M. Am. Soc. C. E., Eng. News, Vol. 21, p. 230-277. (March 16, 1889). (Similar to article in Trans. Am. Soc. C. E., Vol. 20, p. 1).

Excessive Rainfalls Considered with Especial Reference to their Occurrence in Populous Districts.—By R. L. Hoxie. With Discussion. Trans. Am. Soc. C. E., Vol 25, p. 70. (July, 1891). (Gives details of the sewers in Washington, D. C., with plates).

Rates of Maximum Rainfall.—By Arthur Newell Talbot. Technograph, No. 6, University of Illinois. (Abstract of the same, Eng. News, July 21, 1892).

Report of Street Department, Boston, Mass., 1892, p. 116; 1893, p. 123.—(Short discussion of method of sewer design with three very useful diagrams; one showing rainfall rates and curves, one showing run-off curves for different areas and slopes and one showing sewer curves for different slopes and sizes. Prepared by E. S. Dorr).

Storm-Water Drainage.—By Kenneth Allen. In Report of the Sewerage Commission of the City of Baltimore, 1897, p. 94. (The McMath form of the Burkli-Ziegler formula has been used in estimating the storm-water to be cared for).

Rainfall and Run-Off in Relation to Sewerage Problems.—By Walter C. Parmley. Jour. Assoc. Eng. Soc., Vol. 20, p. 204. (March, 1898). (Contains a table showing the intensity of precipitation in inches per hour for different cities).

Notes on Storm-Water Sewers.—Eng. Record, Vol. 38, p. 379. (Oct. 1, 1898). (Editorial on the subject contains a diagram of storm-water run-off prepared by J. H. Fuerties).

Tables of Excessive Precipitations of Rain at Chicago, Ill., from 1889 to 1897, Inclusive.—By Edwin Duryea, Jr. Jour. Western Soc. E., Vol. 4, p. 73. (Feb., 1899). (Tables comparing run-off with precipitation by six formulas). (Discussion has tables showing capacities of Chicago Main Sewers).

Sewer Design.—By H. N. Ogden, Assoc. M. Am. Soc. C. E. (New York: John Wiley & Sons; 1899). (Contains Chapters on excessive rain; proportion reaching the sewers; mathematical formulas, etc.).

The Volume and Velocity of Run-Off Water. (Abfluss-Geschwindigkeit und Quantitat).—By Rudolf Muller. Oesterr, Monatschr, f, d Baudienst. (Sept., 1899). (An exposition of a new method of computing the run-off of rain-water with diagrams showing graphical solution of given cases.

Paper on the ability of the soil in cities to absorb rainfall with special reference to the delay in the run-off of the same. (Ueber die Stadt-Enwasserungsanlagen zu Grunde zu legenden Regenmengen mit besonderer Berucksichtigung der Verzogerung im Abflusse derselben).—Von Eduard Bodenseher. Zeitschrift des Oesterr. Ingenieur und Architekten-Vereines, Vol. 52, p. 257. (Apr. 20, 1900). (Contains table of the run-off).

Storm Flows from City Areas and their Calculations.—By Ernest Wilder Clark. Eng. News, Vol. 48, p. 380 (Nov. 6, 1902). (Contains diagrams showing maximum precipitation in various cities in periods of 10 to 120 minutes).

The Design and Construction of a 50-ft. Brick Arch Culvert Across Rock Creek, Washington, D. C.—By W. J. Douglas. Eng. News, Vol. 48, p. 534 (Dec. 25, 1902). (Calculation of flood discharge in the absence of actual gagings).

Notes on Designing Chicago Sewers.—By C. D. Hill. Jour. Western Soc. E., 1902, p. 435.

The Measurement of Rainfall and Run-Off. (Ueber Regenhohen und Abflussmengen.)—By Prof. Busing. (Gesundheits-Ingenieur). (Feb. 28, 1903). (A discussions of the probable rainfall in cities and the volumes of storm-water to be provided for by the sewerage systems).

Principles for the Design of a Municipal Sewerage System. (Die Grundlagen zur Berechnung von Stadteentwasserungsanlagen).—By H. Heyd. (Gesundheits-Ingenieur, Jan. 20, 1905). (A discussion on the fundamental principles for determining the amount of storm-water and sewage to be carried off from a given locality).

The Elimination of Storm-Water from Sewerage Systems.—By David Ernest Lloyd-Davies, Assoc. M. Inst. C. E., Min. Proc. Inst. C. E., Vol. 164, p. 41-67. (1905-06).

Rainfall and Run-Off in Sewer Design.—By Chas. E. Gregory. Munic. Engng. (Sept., 1905). (Extracts from a lecture before the Engng. School of Union College, Schenectady, N. Y., and a discussion before the Am. Soc. of C. E., giving the latest advances in this subject).

Rainfall and the Proportion to be Provided for in the Sewers.—Sewers and Drains for Populous Districts. (p. 27-34). By Julius W. Adams.

SEWER GAGINGS—RESULTS OF ACTUAL EXPERIMENTS.

Capacity for Storm Water.—By Maj. G. J. Lydecker, Eng. Com'r. An. Rep. Eng. Dept., District Columbia, (1885) p. 16. (Some observations made in Washington, D. C., on the rapidity with which storm-water finds its way into sewers. A self-recording gage of the height of water in the sewer was used in the experiments. A capacity of 2 ins. of rainfall an hour found to be none too large for well built up and paved sections).

The relation between the Rainfall and the Discharge of Sewers in Populous Districts.—By Emil Kuichling. Trans. Am. Soc. C. E., Vol. 20, p. 1 (Jan., 1889). (Contains tables on discharge of sewers in different districts of Rochester, also gagings of rainfall and discharge in other cities).

Report upon the Sewerage of the District of Columbia.—By Rudolph Hering, Samuel M. Gray and F. P. Stearns, 1890.

Flood Waves in Sewers and their Automatic Measurement.—By Alva J. Grover. With discussion. Trans. Am. Soc. C. E., Vol. 28, p. 1 (Jan., 1893). (An article on sewer gagings and their relation to rainfall).

The Methods and the Results of Experiments for the Determination of the Velocity of the Flow of Water.—Made under the direction of the Bureau of Surveys, City of Philadelphia. By Charles Jacobsen. With discussion. (Proc. Eng. Club of Phila., July, Sept., 1897). (Gives relative percentage of rainfall which would reach the sewers).

Sewer Gagings at Sheboygan and Milwaukee.—By R. A. Logemann and R. A. Nommensen. Eng. News, May 30, 1901, p. 406.

Ground Water in Sewers.—By J. N. Hazelhurst, M. Am. Soc. C. E. Eng. News, Aug. 27, 1903.

Rainfall and Run-Off. (Regenabfluss und Abflussversorgerung).—By W. Krawinkel. (Gesundheits-Ingenieur, May 10, June 10, 1905). (A study of the surface and underground run-off for various conditions, with data for the design of sewers and drainage channels). Two articles.

Rainfall and Run-Off in Storm Water Sewers.—By C. E. Gregory. Trans. Am. Soc. C. E., Vol. 58, 1907.

THE COST OF LAYING 6 AND 8-IN. WROUGHT-IRON SCREW-JOINT PIPE.

By E. E. HARPER.*

During the summer of 1907 the writer had occasion to lay about 7,000 ft. of 8-in. and 4,000 ft. of 6-in. well casing, to be used as an air line for compressed air at 80 to 90 lbs. pressure. Very careful attention was given to the cost of this work. In view of the scarcity of similar cost

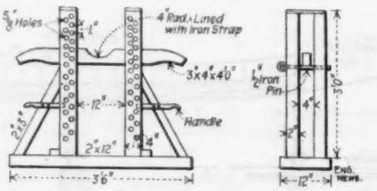


Fig. 1. Jacking Used for Holding Outer End of Screw-Joint Wrought-Iron Pipe in Position While Connecting It Up.

data, if any be available, I submit my itemized figures to the profession through your columns.

The work was all performed by common labor, none of the men being experienced in pipe-laying. The accompanying tables give the cost per 100 ft. of laying the 6-in. and 8-in. casing. The illustrations show the devices used to facilitate the work.

*Of Ward & Harper, Civil and Mining Engineers, Sturgis, Ky.

*PREMIUM SCHEDULE FOR LAYING S.N. PIPE.

After working four days the following schedule was adopted and the men were paid according to the number of joints laid per day. No deduction was made for U-bends or fittings.

A U-bend counted as 1 joint.

For every man added to the crew, add 3 joints.

For every man less than the crew, deduct 3 joints.

In counting joints, any odd joints carried forward and counted next day. Thus, 28 joints gives 10 hrs. time with 2 joints to credit of crew for next day's work.

Schedule for 5 men, 1 foreman, 1 waterboy.

22 joints (or less)	10 hrs. time for crew
26 joints	11 hrs. time for crew
29 joints	12 hrs. time for crew
32 joints	13 hrs. time for crew
35 joints	14 hrs. time for crew
38 joints	15 hrs. time for crew
41 joints	16 hrs. time for crew
44 joints	17 hrs. time for crew
47 joints	18 hrs. time for crew
50 joints	19 hrs. time for crew
53 joints	20 hrs. time for crew

*Same schedule used in laying 6-in. pipe, except that size of crew was reduced by one man.

The greatest cause of delay in laying screwed pipe is the difficulty in getting each successive length of pipe into line and keeping it there until the first threads take hold and the pipe begins to screw together. To overcome this difficulty a cradle for supporting the pipe at the joint, a jack for adjusting and supporting the outer end of the pipe and a straight-edge for lining the pipe were devised. The cradle holds the threaded end of the pipe in position to enter the sleeve coupling on the last joint laid; the jack allows both vertical and horizontal adjustment of the joint of pipe; and the straight-edge shows when the pipe is in line ready to screw together. The cradle was simply a wood block, 8 × 8 ins. × 24 ins. in length, with a groove having a 4-in. radius cut in its top. The jack is shown by Fig. 1 and the straight-edge by Fig. 2. The movable block on the straight-edge (Fig. 2) is necessary because it is almost impossible to make a 12-ft. straight-edge that will remain true for more than a day.

These devices saved fully 50% over the crude and unsatisfactory method of using blocks to

COST OF LAYING 6 AND 8-IN. (OUTSIDE DIAMETER) WROUGHT-IRON AIR-SUPPLY PIPE FOR THE FEDERAL LEAD CO., FLAT RIVER, MO.

6-IN., FROM NO. 9 SHAFT TO NO. 10 SHAFT.

Item.	Cost per 100 ft.	Remarks.
1 Size and kind	....	6-in. O.D. lap-welded casing. Weight, 15 lbs. per ft. Av. length of joints, 20.1 ft. Laid with sleeve couplings, 11½ threads per in. Flange union every 150 ft. U-bends for expansion every 500 ft.
2 Length	....	4118 ft.
3 Surveying and superintendence	$0.70	
4 Cost of laying	3.137	Actual cost of connecting pipe. Cost per joint, 62.5c.
5 Av. ft. laid per day	....	*588.2 = 29.3 joints.
6 Crew and wages	....	Av. Crew, 4.85 men at 20c.; 1 foreman at 30c.; 1 waterboy at 10c. See premium schedule.
7 Clearing right-of-way	0.327	Small brush, etc. Cut line 10 ft. wide.
8 Hauling and distr.	1.578	Av. haul, 3,000 ft. bad roads, steep and rough. Includes loading pipe on cars and unloading and hauling and distr. Teams and drivers $3 per day. Incl. 7 U-bends.
9 Blocking to grade	0.116	Includes temporary blocking and bending pipe in 5 places by building fires on same.
10 Cost of bents	0.45	Carpenter work only. About 20 bents. Av. 3 ft. height. 4 × 6-in. stuff.
11 Anchors for U-bends	2.29	8 piers, ¾ cu. yd. each, $12 each, incl. bolts and clamps. (Est. cost.)
12 Painting	0.90	Paint, $1 per gal. Labor, 20c. per hr. Pipe cleaned with wire brushes. Given 1 coat mineral paint. (Est. cost.)
13 Tools	0.10	Shopwork, depreciation.
14 Testing	0.30	(Est. cost.)
15 Total cost of laying	9.898	
16 Cost of pipe	....	
17 Weather, etc	....	Weather fair except 1 day. Men worked in rain 4 hrs. and laid 22 joints. Men walked 2½ to 3 miles to and from work. Time, Aug. 15 to 24, '07.

*Best day's work, 1065 ft. or 53 joints by 6 men, incl. foreman, or 177.5 ft. each, actual time 9 hrs. (See 17); 2d best day's work 883 ft. or 44 joints by 6 men, incl. foreman, or 147.1 ft. each, actual time 9 hrs. (See 17); poorest day's work 120 ft. or 6 joints by 6 men, incl. foreman, or 20 ft. each, actual time 9½ hrs. (See 17). No excuse except poor judgment and bad management of foreman.

8-IN., FROM NO. 3 MILL TO NO. 9 SHAFT.

Item.	Cost per 100 ft.	Remarks.
1 Size & kind of pipe	....	8-in. O. D. Lap-welded casing. Weight, 20 lbs. per ft. Av. length of joints, 19.15 ft. Laid with sleeve couplings, 11½ threads per in. Flange union every 150 ft. U-bends for expansion every 600 ft.
2 Length of line	....	7,101 ft.
3 Surveying and superintendence	$1.00	
4 Cost of laying	3.58	Actual cost of connecting pipe. Cost per joint, 69.3c.
5 Av. ft. laid per day	....	*503.6 per day of 10 hrs. = 26.3 joints.
6 Crew and wages	....	Av. crew, 5.92 men at 20c. per hr.; 1 foreman at 30c.; 1 waterboy at 10c. per hr. See premium schedule.
7 Clearing right of way	0.187	Small brush, etc. Cut line 10 ft. wide.
8 Hauling and distr. pipe	1.0321	Incl. 12 U-bends, which cost $1 each to haul. Team and drivers at 30c. per hr.; laborers, at 20c.; foreman at 30c. per hr.
9 Blocking to grade	1.11	Temporary Supports. Bents put in afterward.
10 Cost of bents	1.0687	Carpenter work only. Abt. 80 bents of 4 × 6-in. stuff from 1 to 16 ft. high; av. 6 ft. high; spaced 30 ft. apart.
11 Anchors for U-bends	2.535	12 piers at 1 cu. yd. each, $15 each, incl. bolts and clamps.
12 Painting	1.20	Paint, $1 per gal. Labor, 20c. per hr. Pipe cleaned with wire brushes. Given 1 coat mineral paint.
13 Tools	0.1022	Shop work, depreciation, etc.
14 Testing	0.388	Includes laying & connecting 200 ft. of 4-in. pipe to pump line. Tested to 110 lbs. hydraulic pressure. Leaks developed at two tees in line, and these were repaired. Line tested again and found O. K.
15 Total cost of laying	12.208	Not including cost of pipe.
16 Cost of pipe	76.40	F.O.B. McKeesport, Pa., 76c.; frt. McKeesport to Flat River, 60c.
17 Weather, etc	....	July 2, '07, to Aug. 5, '07, weather hot and sultry most of time. Thermometer from 85° to 100° in shade; ave., 90°.

*Best day's work, 613 ft. or 32 joints by 6 men, incl. foreman, or 102 ft. each; 2d best day's work, 675 ft. or 35 joints by 7 men, incl. foreman, or 96.4 ft. each; poorest day's work, 380 ft. or 20 joints by 7 men, incl. foreman, or 54 each.

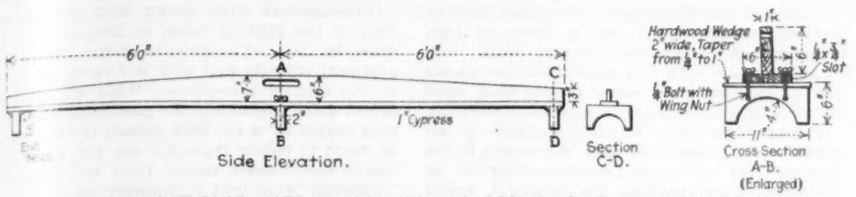


FIG. 2. STRAIGHT-EDGE USED IN CONNECTING SCREW-JOINT WROUGHT-IRON PIPE.
(By using a straight length of pipe the straight-edge may be quickly adjusted or tested.)

hold the pipe in line. There was no straining and lifting to hold the pipe in place, and as the pipes were started together straight there were no stripped threads and bad joints, and the pipe made up so easily that one man with a pair of 3-ft. tongs often screwed an 8-in. pipe half way up; it was then completed by four men using two pairs of tongs with 8-ft. handles.

The threads, both male and female, were cleaned with wire brushes. Dixon's pipe joint compound was used on all screwed joints. Ring gaskets of 1-16-in. rainbow packing were used on flange joints (see table), the gasket being pasted to one flange with coal-tar roofing paint, which held it in position while the joint was being made.

The premium schedule (see foot note to table) allowing overtime if more than a certain amount of pipe was laid per day, proved quite satisfactory and kept the men hustling.

HYDRAULIC DREDGE FOR RECLAIMING LAND FOR LINCOLN PARK, CHICAGO.

An extension of Lincoln Park, Chicago, is being made by filling in a large area north of the present park by means of suction dredges excavating material from the bed of Lake Michigan and pumping it ashore through a pipe line. The filled area will extend about a mile along the shore, with a width of 1,500 ft., and will require about 4,000,000 cu. yds. of material. The work is partly in water 18 ft. deep and the lake front of the park is defined by a rough breakwater or bulkhead wall made of stone brought in barges from the spoil banks of the Chicago drainage canal. The material dredged is the stiff blue Chicago clay, more or less mixed with gravel and stones. Different methods of working were considered by the Lincoln Park Commission, and as the result of investigations made by the late Reuben H. Warder, Secretary and Superintendent to the Commission, it was decided to adopt hydraulic dredging and pumping as the most economical and the most suited to the conditions. The Commission also decided to do the work itself. In view of the character of the material to be handled, and the fact that the work is in deep water, exposed to the sudden and severe storms of Lake Michigan, a suction dredge was specially designed for this work by Mr. A. W. Robinson, M. Am. Soc. C. E., of Montreal, Canada. It was built by the Atlantic Equipment Co., of New York, and was in service last season.

Fig. 1 is a general view of the dredge, which is named the "Francis T. Simmons," after the President of the Lincoln Park Commission. This view also shows the floating discharge pipe, and (in the distance) the elevation of this pipe on a scow so as to pass over the stone bulkhead wall marking the front of the future park. The hull is of steel, 148 × 38 ft., and 10½ ft. deep. A superstructure or deck house extends over the greater part of the length, and above the forward end of this is a pilot house or operating house from which all operations are controlled. Hinged to the bow of the hull is a steel ladder frame carrying the suction pipe. The outer end of the ladder is suspended from a boom which is stayed to an A-frame on the hull. Backstays are attached to the head of the A-frame. The ladder is 40 ft. long, and the machine will dredge to a depth of 32 ft. At the stern is a gallows frame from which the anchor spuds are suspended by tackle, the lines being led to the engine drums.

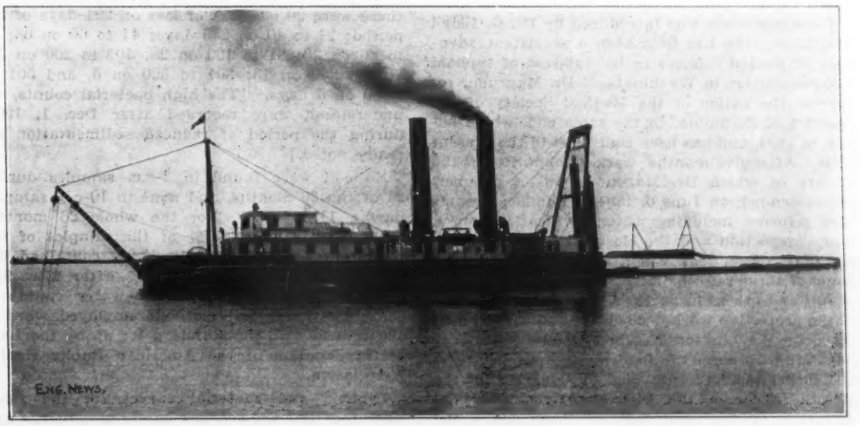


FIG. 1. SUCTION DREDGE "FRANCIS T. SIMMONS" FOR THE LINCOLN PARK COMMISSION, CHICAGO.
A. W. Robinson, M. Am. Soc. C. E., Montreal, Designer; Atlantic Equipment Co., New York, N. Y., Builders.

The centrifugal dredging pump has 30-in. suction and discharge openings, and is driven by a triple expansion engine of 1,200 HP. Steam is supplied by two Scotch or marine boilers 11½ × 18 ft., each having four furnaces. The suction pipe is 30 ins. diameter, with a flexible joint at the hull connection. The outer end is fitted with a powerful cutter, which revolves around the mouth of the pipe and is driven (by means of shafts and gearing) by a tandem compound engine of 300 HP. The cutter is a steel casting 9 ft. diameter and weighing about 9 tons. It is formed with eight blades which curve outward and backward from the inner side, and are bolted at the rear to a circular steel plate. The blades are fitted with renewable cutting edges of hard steel. The pump equipment has ample power for handling the large slices of clay removed by the cutter blades, and the discharge pipe is not liable to clog unless the suction is fed too fast. During the season of 1907 the machine worked 122½ days (of 24 hours), and the total excavation was 457,242 cu. yds. The maximum excavation per hour was 866 cu. yds., and the average was 426 cu. yds. per hour. The material as pumped averaged 10% of solid matter.

The discharge pipe line is composed of 30-in. pipe in lengths of 93 ft. 6 ins. Instead of being fitted with rubber sleeve joints and supported at intervals by floats or pontoons, the sections have ball and socket joints, and each length is supported between two 33-in. cylindrical steel pontoons about 100 ft. long. The connections between the sections are provided with spring attachments to relieve the joints from the longitudinal stresses due to surging or wave motion. These are fitted with drawbar springs, arranged somewhat as in the draft rigging of railway cars. Other springs control the lateral movements at the joints. The connection of the fixed discharge pipe on the dredge with the floating discharge pipe can be instantly disconnected by means of a lever. This arrangement of long pontoons connected to form a continuous support for the pipe line was adopted in order to secure steadiness in the pipe, and has proved very satisfactory in rough weather. Fig. 2 is a view showing the discharge pipe and also the spud frame on the stern of the dredge. A similar pipe line for the Canadian dredge "Israel J. Tarte" (also designed by Mr. Robinson) was illustrated in our issue of July 28, 1904.

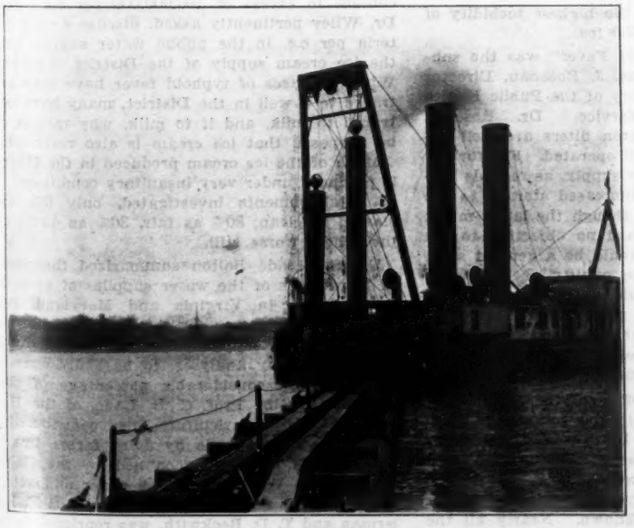


FIG. 2. PIPE LINE AND PONTOONS FOR DISCHARGE FROM SUCTION DREDGE, SHOWING ALSO THE GALLOWS FRAME FOR HANDLING THE SPUDS.

TWO THOUSAND-VOLT DIRECT-CURRENT LOCOmotives are used on a meter-gage mine railway of about 9 mi. length at Maizières, Lorraine. The gradients of the railway are 2% and 3% in parts, and ore trains of 250 to 300 tons are hauled over it. Hitherto the service has been performed by steam locomotives, but now electric traction has been installed. Direct-current motors were chosen because it was possible to get motors of larger capacity into the available width than with other systems. The locomotive has two four-wheel trucks, and each of the four axles carries a motor, whereby a total hourly capacity of something over 600 HP. was obtained. The motors are geared. They are connected permanently two in series and are fed by overhead wire carrying 2,000 volts pressure, with rail return. The current collector is a side-contact sliding conductor of the Oerlikon type. The line is fed from a 600-KW. converter station at either end of the line. The locomotives have 49-in. drivers and weigh 60 tons each. The motors have commutating poles.

WATER SUPPLY AND TYPHOID FEVER AT WASHINGTON, D. C.

A symposium on water supply and typhoid fever and various related subjects was presented before the Medical Society of the District of Columbia on Wednesday evening, Feb. 19. It is gratifying to note that, taken as a whole, the papers and discussions of the evening practically cleared the slow-sand filters at Washington of the suspicion cast upon them by some, that they had not done all that could be reasonably expected of them to reduce typhoid fever in Washington since they went into full use in November, 1905. (For a description of the filters and exhaustive reports on the Washington typhoid situation, see Eng. News, Nov. 8, 1906; and for an abstract of the elaborate report on Washington typhoid made by the U. S. Public Health and Marine-Hospital Service, see Eng. News, July 11, 1907). In addition to the facts brought out in the symposium, we present at the close of this report of the meeting some typhoid figures for Washington which have kindly been supplied us at our request, as noted farther on.

The symposium was introduced by Dr. G. Lloyd Magruder, who has long been a persistent advocate of needed reforms in the interest of typhoid fever reduction in Washington. Dr. Magruder reviewed the action of the Medical Society of the District of Columbia, to the same end, which began in 1894, and has been continued to the present time. After five months' work a committee of the society of which Dr. Magruder was a member,, recommended, on June 6, 1894, a number of sanitary reforms, including water purification works, dairy inspection and the closing of a large number of shallow wells which existed at that time. Some of these reforms have since been carried out in full and others in part. A considerable portion of the protective work needed to put down typhoid fever in Washington has been made difficult or impossible through the inadequate appropriations and inspection force provided by Congress for the Health Officer of the District. Even after the agitation of the past two or three years the Health Officer is still dependent upon various government departments for bacterial examinations.

In concluding his introductory remarks, Dr. Magruder stated:

> It will be shown this evening that clear, absolutely safe water is furnished direct from the taps the greater portion of the year, that the slightly turbid and reasonably safe water can readily be made clear and absolutely safe; that the prevalence of typhoid fever, diphtheria, scarlet fever and tuberculosis can be diminished and that a marked diminution in the diseases and deaths of infants may be secured. This will be accomplished by furnishing the District Commissioners additional authority for the regulation of dairy products—milk, cream, ice cream and butter—a bacteriological laboratory and additional inspectors for the health department. Many of the dangerous conditions at dairy farms could, and would, at a trifling cost, be promptly remedied were the farmers properly instructed by competent inspectors.

Following Dr. Magruder's remarks, Major Spencer Cosby, Corps of Engineers, U. S. A., described the Washington water supply. Among the points made by him, that need be mentioned here, were the facts that taking stream flow and detention in reservoirs into account, a period of seven to ten days elapses between the time the water of the Potomac River passes the last important source of pollution and reaches the consumer. When the water of the river is very turbid the gates at the head of the aqueduct are shut. Sedimentation alone has reduced the turbidity by 63% for a long period, and for a whole year it has reduced the bacterial contents by 87%. Some 45% of the samples of crude water have contained the colon bacillus, against only 2.8% of filtered water samples. Filtration reduced the average bacterial contents of the water from about 600 to 30 per c.c., and at the same time reduced the turbidity to a maximum of 20. The figures just cited are for the first two years of operation. In December, 1907, the examination showed a high turbidity at a period when the Delecaria Reservoir was out of use for repairs. At the same time the bacteria were comparatively high, but these conditions were not accompanied by an increase in typhoid fever.

As many of our readers are aware, all the engineers who have had to do with both preliminary investigations and the construction and operation of the Washington filters have recommended coagulation for use at times of high turbidity. A year's experimental work with coagulation and with two rapid filters has shown that coagulation gives better results than rapid filtration as a preliminary to slow sand filtration.

If coagulation were adopted, sulphate of alumina would probably be used, introduced in the aqueduct just before the Delecaria Reservoir, so the water, after receiving the coagulant, would pass through six miles of aqueduct and two reservoirs, besides the filtration plant, before reaching consumers. Congressional authority must be secured before coagulation can be adopted.

Mr. F. F. Longley, Chief Chemist of the filtration plant, discussed the Washington filters and the quality of their effluent. He emphasized the danger of ignoring other causes of typhoid fever than those connected with the water supply, and said that it was logical to conclude that too much blame for typhoid had been placed in the past on the muddy Potomac. Records of the bacterial content of filtered water during the period from Nov. 1, 1905, to Feb. 1, 1908, showed that there were 20 bacteria or less on 291 days of the period; 21 to 40 on 245 days; 41 to 60 on 63; 61 to 80 on 30; 81 to 100 on 28; 101 to 200 on 29; 201 to 300 on 13; 301 to 500 on 5, and 501 to 1,000 on 5 days. [The high bacterial counts, we understand, were recorded after Dec. 1, 1907, during the period of reduced sedimentation already noted.]

No coli were found in 1-c.c. samples during 21 of the 26 months, and none in 10-c.c. samples during 11 months. For the whole 26 months, with 1-c.c. samples, 37% of the samples of the river water contained coli, 11.9% contained the germ after sedimentation, 1.5% after filtration, and 1.1% of samples of tap water contained coli. The analytical methods employed were in accordance with the standard set up by the Laboratory Section of the American Public Health Association.

Typhoid and bacterial curves for the years 1906 and 1907 showed low bacterial contents and high typhoid during each summer, but in December, 1907, when for the causes already stated bacteria were high, typhoid was low. The operations of the plant show the advantages that would result from the use of a coagulant. Besides reducing turbidity, the color would be reduced.

The objections that have been urged against the use of sulphate of alumina were discussed by Mr. Longley in some detail. Briefly, he disposed of them by stating that the minimum amount of carbonates in the Potomac water is 31 parts per 1,000,000, which is sufficient to neutralize 516 lbs. of sulphate of alumina per 1,000,000 gals. of water. The maximum amount of coagulant needed for the highest turbidity of Potomac water is only 360 lbs.

"The Causes of Typhoid Fever" was the subject of a paper by Dr. M. J. Rosenau, Director of the Hygienic Laboratory of the Public Health and Marine-Hospital Service. Dr. Rosenau stated that the Washington filters are well designed, well built and well operated. For further improvement in the water supply, as regards turbidity, he would prefer increased storage to the use of a coagulant, even though the latter might be less expensive. He had no objection to the use of alum, but there would be a general sanitary improvement in the quality of the water due to storage that would not follow from the use of a coagulant. By providing a total of 20 days storage, the gates at the head of the aqueduct could be closed so as to exclude the water at all periods of high turbidity.

During the summer of 1907 there were 200 typhoid cases less than in the summer of 1906. There are some reasons, Dr. Rosenau stated, for supposing that this decrease was due to filtered water. The subject, however, is a difficult one and the exact part played by Potomac water will probably never be known. Nearly all the shallow wells in the District have been closed, but no case of typhoid fever was ever traced to one of these wells, although it is well known that they were polluted.

Investigations have shown that milk caused 10% of the typhoid cases in 1906, and 7% of those in 1907. The records of the Health Department indicate that milk is a recurrent factor in the typhoid prevalence. While awaiting efficient dairy inspection, the pasteurization of the milk supply after the milk reaches the city would do much to reduce typhoid fever, and would also reduce tuberculosis, scarlet fever and dysentery.

Contacts were held responsible for 7% of the typhoid cases in 1906, and 19% in 1907. In Germany, typhoid cases are isolated and the disease is considered and treated as falling in the dangerous communicable class.

Imported typhoid was held accountable for 15% of the cases in 1906, and 25% in 1907.

Approximately, the typhoid board accounted for 30% of the typhoid cases in 1906 and 50% of those in 1907, the chief ascertained causes being, as already indicated, milk, contact and importations.

The "Dangers of Domestic Filters, of Water-coolers and of Ice Cream" were presented in an interesting and forceful manner by Dr. H. W. Wiley, Chief of the Bureau of Chemistry, U. S. Department of Agriculture. As a preliminary, Dr. Wiley stated that, in general, analyses of the city water made under his direction agreed with those reported by the chemist of the filtration plant in showing a very low bacterial content. Experiments with a freshly sterilized domestic filter showed that by this means the very low bacterial content of the tap water (an average of 13 or so) was reduced some 55%, but evidence was accumulated which showed that domestic filters when not properly cared for do more harm than good, particularly as giving rise to a false sense of security.

A study of the water as found in water coolers in various bureaus of the Department of Agriculture showed that the nearly sterile tap water made available by the filtration plant was often badly contaminated by the ice used in the coolers. Some of this contamination was not due to the ice itself, but to dirt gathered by it during the process of handling.

At the present time, Dr. Wiley stated, typhoid fever in Washington connot be attributed to the public water supply, except as the water is contaminated by dirty ice and ill-kept water coolers. There may, perhaps, be some lingering typhoid germs in the water mains, but the public water supply as a whole is a safe beverage.

Far different conclusions were drawn from studies of the ice cream supply of the District. Examinations of the 250 samples of ice cream showed an average bacterial content of 26,000,000 per c.c., and only 5 of the 263 samples contained less than 500,000 per c.c. The maximum was 365,000,000, and 16 samples showed a bacterial content in excess of 100,000,000 per c.c. Why, Dr. Wiley pertinently asked, discuss 4 or 5 bacteria per c.c. in the public water supply, when the ice cream supply of the District is so bad? While no cases of typhoid fever have ever been traced to a well in the District, many have been traced to milk, and if to milk, why may it not be supposed that ice cream is also responsible?

Much of the ice cream produced in the District is produced under very insanitary conditions. Of 53 establishments investigated, only 6% were classed as clean, 30% as fair, 36% as dirty, and the others worse still.

Dr. B. Meade Bolton summarized the results of inspections of the water supplies of some 300 dairy farms in Virginia and Maryland, from which milk is drawn for the supply of the city. Many of these supplies were found, both by inspection and by analyses, to be badly contaminated, and a considerable percentage of them showed B. coli. Prof. C. B. Lane, of the U. S. Department of Agriculture, also contributed to the bad showing made by dairy farms, but his observations were not confined to the dairies tributary to Washington. [A paper on bacteria in dairy wells near Washington, by Karl F. Kellerman and T. D. Beckwith, was reprinted in our issue of Feb. 7, 1907.]

An important paper on "The Early Diagnosis of Typhoid Fever" was contributed by Dr. L. L. Lumsden, of the Public Health and Marine-Hos-

TABLE I.—STATEMENT SHOWING THE PREVALENCE OF TYPHOID FEVER IN THE DISTRICT OF COLUMBIA, BY MONTHS AND BY YEARS, SINCE JULY 1, 1902.

Month.	Cases reported. 1902.	1903.	1904.	1905.	1906.	1907.	Deaths reported. 1902.	1903.	1904.	1905.	1906.	1907.
January	....	84	17	31	27	53	....	8	5	10	6	7
February	....	42	23	10	21	32	1	6	5	2	4	6
March	....	35	42	24	18	25	12	8	7	5	5	4
April	....	40	25	24	35	28	9	7	10	7	4	6
May	....	63	32	28	44	a37	12	5	8	1	10	7
June	....	57	41	40	58	36	10	3	8	3	9	2
July	129	121	101	133	181	82	21	16	14	14	21	10
August	327	185	225	314	293	191	39	25	24	30	32	18
September	289	138	214	212	150	187	25	18	27	23	20	17
October	245	148	143	b150	171	149	32	19	14	27	28	19
November	156	88	105	80	82	87	19	8	9	13	19	11
December	128	54	38	51	46	40	20	14	10	7	4	7
Total	1,274	1,055	1,006	1,097	1,126	945	200	137	141	142	162	114

aShallow wells ordered closed by the Commissioners of the District of Columbia, May 21, 1907.
bFiltration of entire water supply begun in October, 1905.

TABLE II.—REPORTED CASES AND DEATHS FROM TYPHOID FEVER IN THE DISTRICT OF COLUMBIA, BY MONTHS, DURING THE CALENDAR YEAR 1907, AMONG THE WHITE AND THE COLORED POPULATION.

	Deaths. White.	Colored.	All.	Reported cases. White.	Colored.	All.
January	6	1	7	46	7	53
February	3	3	6	28	4	32
March	2	2	4	23	2	25
April	3	3	6	20	8	28
May	5	2	7	31	6	37
June	1	1	2	30	4	34
July	6	4	10	61	21	82
August	13	5	18	145	46	191
September	13	4	17	146	41	187
October	12	7	19	102	47	149
November	8	3	11	68	19	87
December	5	2	7	35	5	40
Total	77	37	114	735	210	945
Calendar year 1906	82	80	162	796	330	1,126

pital Service. He emphasized the dangers to the public due to infection by typhoid patients from the beginning of typhoid sickness and even from the beginning of incubation. He also described a direct bacterial count of the typhoid germs from the blood of patients, which can be made in 24 hours. Mr. J. H. Kastle, Chief of the Division of Chemistry, U. S. Public Health and Marine-Hospital Service, presented some of the dangers due to chronic typhoid-bacillus carriers, and described the use of a special drug for destroying typhoid bacilli in the gall bladder. Surgical work at Johns Hopkins University was cited by Mr. Kastle to show that the gall bladder may contain typhoid germs in the case of persons with no previous history of typhoid fever.

The session having extended from 8 o'clock until past 11, and having sufficed only for the presentation of the papers listed on the program, discussion was postponed until the next meeting of the society.

Through the kindness of Dr. Wm. C. Woodward, Health Officer of the District of Columbia, we are enabled to present herewith (Table I.) typhoid fever statistics for the District, by months, from 1902 to 1907, inclusive, and also (Table II.) the cases and deaths for 1907, distributed among the white and the colored population of the District. In regard to Table II., Dr. Woodward has written to us as follows:

An interesting feature of last year's typhoid fever record, the exact significance of which is not yet apparent, is the remarkable decrease in the number of reported cases and in the number of deaths among colored people. Comparing figures for 1906 with those for 1907, it appears that among white people the reported cases decreased from 796 to 735, while among colored people, the decrease was from 330 to 210. The decrease in the number of deaths among white people during the same period was from 82 to 77, and among colored people from 80 to 37.

The number of cases of typhoid fever reported in January, 1907, Dr. Woodward informs us, was 53, and the number reported during January, 1908, was only 38. Editorial comment on the subject discussed in this article is printed elsewhere in this issue.

A TUNNEL EXCAVATING MACHINE FOR SOFT MATERIAL.

A tunneling machine for excavating in clay and other soft material has been invented by Mr. Geo. W. Jackson, of Chicago, who has already built about 60 miles of deep-level tunnels (5 × 7½ ft. in section) for the Illinois Tunnel Co., and is building extensions of this system as well as some larger water tunnels for the city of Chicago. All this work is in the stiff blue Chicago clay, and in hand work the material is excavated mainly by the use of drawknives. A special feature of the new machine is that its cutters are of similar shape to these tools. Fig. 1 is a view of a working model, and Fig. 2 shows the machine is now being built at the works of Geo. W. Jackson, Inc., Chicago.

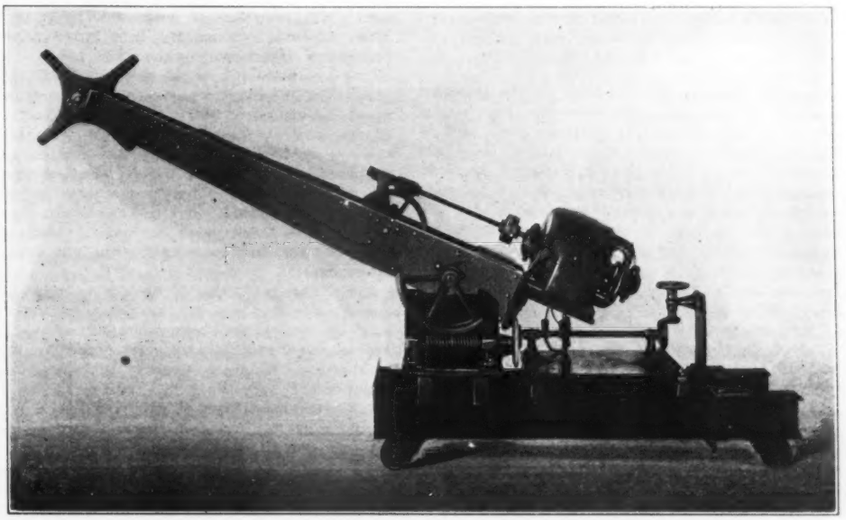


FIG. 1. MODEL OF TUNNEL EXCAVATOR FOR SOFT MATERIAL.

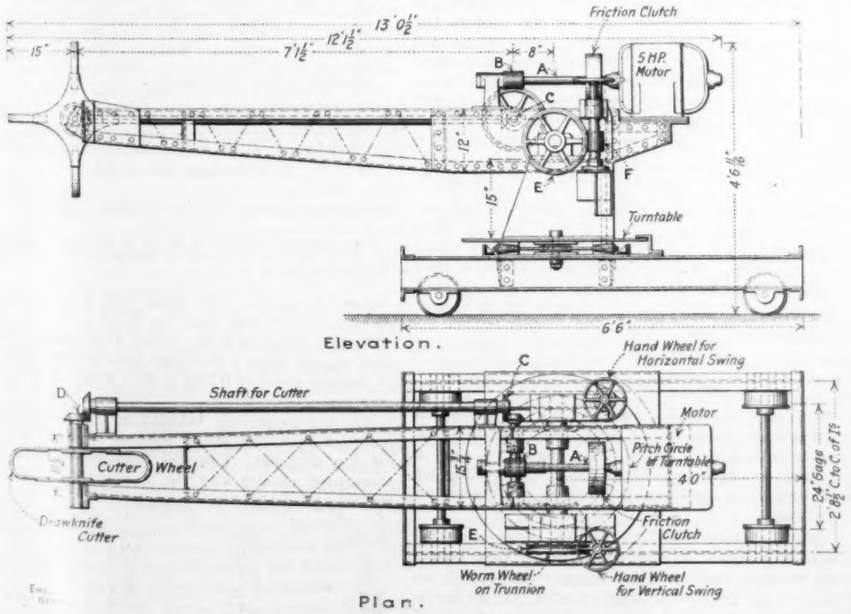


FIG. 2. TUNNEL EXCAVATOR FOR SOFT MATERIAL.
Geo. W. Jackson (Inc.), Chicago, Builders.

The general design and the machinery equipment are very simple. A steel car mounted on wheels carries a turntable having a frame to which the boom of the machine is pivoted. At the head of the boom is a cutter consisting of two four-armed star wheels, each pair of arms carrying a drawknife. An electric motor mounted at the heel of the boom drives (through a friction clutch) a shaft, A, which by worm gearing, B, and bevel gearing at C drives a shaft extending along the boom. This in turn operates the cutter by bevel gearing at D. On one of the trunnions of the boom is a worm wheel, E, operated by a worm F; this latter is fitted with a hand wheel, and by this means the boom can be revolved through a large vertical angle. In the model the worm is horizontal and operates a geared sector, but in the machine as now being built the worm is vertical, and gears with a wheel on the trunnion. At the same time the turntable gear allows the boom to be revolved in a horizontal plane, this gear being operated by another hand wheel. The machine may be fed forward by jacks or by a motor and gearing. The material excavated by the cutter will fall upon a conveyor extending under the boom (but independent of the tunneling machine), and this will deliver it to a second conveyor passing under the machine and discharging into the cars.

The machine can be adapted to coal mining by the use of suitable picks on the cutter wheel. It can also be converted into a loading machine for handling various materials by fitting buckets upon the wheel.

SPIRAL ANCHORAGE FOR CONCRETE REINFORCEMENT.

By DANIEL B. LUTEN.*

Anchoring the reinforcement is not an essential feature of most concrete structures. The reinforcement in ordinary beams is stressed a maximum near the middle of its length and in decreasing intensity towards its ends, and the increments of stress are readily resisted by the bond between the concrete and steel. In such structural members there is then no occasion for any anchorage except possibly a short extension of the reinforcement beyond the region of tension.

There are, however, occasional uses of reinforcement in concrete where the tension in the reinforcement is very nearly constant and must

Fig.1.

be transmitted to the concrete at the extreme ends of the member. Thus in Fig. 1 is represented a beam subject to uniform loads and of depth increasing towards the middle of the span. In such a beam there is practically no diagonal tension, and the reinforcement should be concentrated along the lower edge throughout the span. The tension in the reinforcement being nearly constant, the stress to be transmitted from the steel to the concrete at the ends of the members is excessive and must be provided for by some unusual method.

There are numerous other instances that might be cited as requiring more than ordinary precautions for anchoring the steel, such as cantilevered brackets, tension members embedded in

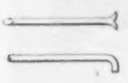


Fig.2.

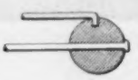


Fig.3.

concrete, and beams of great depth intended for extreme loads.

It has been quite usual to employ anchoring devices in the former class of reinforcements which really require no anchorage. But such devices have generally been of an inefficient form, whose lack of merit would have been quickly proved had they been applied to such a test as required in the latter cases mentioned above. The methods of anchorage that have usually been employed are illustrated in Fig. 2. The end of the reinforcing rod is split open, upset or turned squarely sidewise. These methods have been used extensively in beams, but if they have proved successful it has been due to the adhesion of the concrete to a length of lightly stressed bar rather than to the anchorage provided by these ends. For if the full strength of the bar were to be exerted at these ends the concrete would crush in compression at

Fig.4.

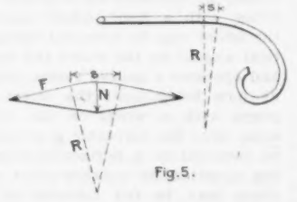


Fig.5.

the shoulder. For similar reasons a deformed bar cannot provide a secure anchorage, although it may require a shorter length to develop its strength through adhesion, than a smooth bar. A washer bearing against a nut on a threaded end would be satisfactory (although very expensive) provided assurance could be had that the nut and washer would bear directly against each other without a film of intervening cement. Such a layer of cement would crush before the strength of the rod could be developed. These types of anchorage provide a bearing surface to the concrete that can hardly exceed the area of the reinforcing member itself. Since the tensile strength of steel is upwards of ten times as great as the crushing strength of concrete, such methods of anchorage cannot develop more than 10% of the strength of the reinforcement, not sufficient to justify their expense.

A comparison with similar anchorages of steel in wood will show a better solution. Let us assume that it be desired to attach a steel rod to a wooden post. In Fig. 3 the rod is shown passing through the post and with an upset end bearing against the opposite surface, and in another instance with a right-angled turn inserted in the post at one side. Clearly the upset end or the split end would be ineffective without a metal bearing plate, since a slight pull on the rod would crush the wood under the expanded end. The same is true of the end bent at right angles, which has even less to commend it. It is possible, however, to modify this latter form to provide a satisfactory anchorage for the steel rod in the wooden post, as shown in Fig. 4, provided the diameter of the post is sufficiently great as compared with that of the rod. All that is necessary to develop the full strength of the rod is to wrap it a sufficient distance about the post. This same principle may be applied in anchoring a steel tension member in concrete, with this advantage, that the curvature may be increased as the tension decreases, resulting in a spiral of radius decreasing from the point of tangency.

Fig. 5 illustrates this form of anchorage. The radius of curvature at the point of tangency may be determined approximately as follows: Assume two radii drawn subtending an arc s as shown to a larger scale in the same figure. The force F represents the tension in the rod, which will equal $\frac{1}{4}\ \pi\ d^2$ 16,000, where d is the diameter of the rod and 16,000 lbs. per sq. in. is assumed as safe tension allowable in the steel. The force N is the allowable normal component of this tension and is equal to 500 sd, 500 lbs. being assumed as the safe allowable compression on concrete. By similar triangles,

$$R/s = F/N. \quad \text{Or } R = sF/N = \frac{4{,}000\ \pi\ sd^2}{500\ sd} = 25\ d.$$

That is to say, the radius of curvature at the point of tangency must not be less than 25 diameters of the reinforcing rod, or the concrete will be in danger of crushing.

Assuming the coefficient of friction of steel on concrete at 4/10 (although it is doubtless much greater), then the resistance to slip due to the normal pressure of 500 sd lbs. is 200 d lbs. per inch of length, neglecting adhesion altogether. Since by increase of curvature the normal pressure may be maintained at 500 sd lbs., the length of spiral required to anchor the rod will be

$$\frac{4{,}000\ \pi\ d^2}{200\ d} = 63\ d.$$

That is to say, with adhesion neglected and depending on friction alone, a rod may be thus anchored by a spiral of length equal to about 60 diameters. If in addition it be assumed that adhesion amounts to 100 lbs. per square inch of surface of rod, then the length of the spiral is reduced to approximately

$$\frac{4{,}000\ \pi\ d^2}{200\ d + 100\ \pi\ d} = 24\ d.$$

Thus on the above assumptions of allowable stresses, a steel rod of 1-in. diameter may be securely anchored by a 24-in. spiral having a radius at tangency of 25 ins., the radius of curvature of the spiral varying inversely as its length.

Since such a spiral cannot be accurately laid out in practice, it may be desirable to increase its length and to approximate the spiral as nearly as possible, being careful to maintain the radii of curvature at values in excess of those theoretically required.

This form of anchorage was employed in a bowstring girder bridge 40 ft. in clear span, described in Engineering News of May 10, 1900, p. 517. The principles outlined in the present article will govern its application to all cases. The design has not been patented.

*President, National Bridge Co., Indianapolis, Ind.

DIAGRAM FOR COMPUTING RUN-OFF BY McMATH'S FORMULA.

By SAMUEL D. BLEICH.*

Whenever it becomes necessary to make computations for sizes of sewers in an extensive sewerage system, some quick methods are absolutely essential to save time and labor, to lessen the danger of making errors, and to avoid the strain and fatigue resulting from extensive numerical computations. It was for these reasons that a diagram for McMath's formula was plotted on logarithmic cross-section paper. The diagram has not only been used for the determination of run-off directly, but also as a check on results obtained by other methods.

The use of the diagram is simple. Given the drainage area in acres and the general slope of the ground or sewers in per cent., the diagram solves at once the expression

$$A^{\frac{4}{5}}\ S^{\frac{1}{2}}$$

Let $A = 234$ acres and $S = 2.5$ ft. per 1,000 or 0.25%. Then the intersection of these two coordinates is at 95 of an interpolated oblique line. This is the result of

$$234^{\frac{4}{5}} \times 2.5^{\frac{1}{2}}.$$

To obtain the discharge in cubic feet per second, it is necessary to multiply this value by $C\ R$, which is generally a constant. In the design of the

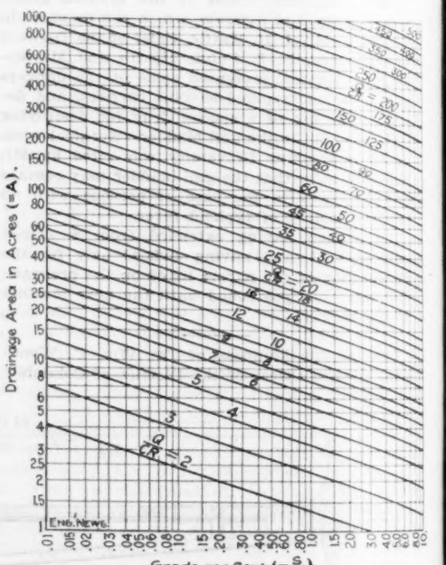


Diagram for Use in Computing Run-Off by McMath's Formula.

(The formula is $Q = C\ R\ A^{\frac{4}{5}}\ S^{\frac{1}{2}}$, in which Q is discharge in cu. ft. per sec.; C ratio of imperviousness; R is rainfall in ins. per hr.)

Brooklyn sewers, the value for C is generally taken at 0.75 and that for R at 3 ins. per hr.; CR then becomes 2.25. Using this value for CR, the run-off from a drainage area of 234 acres and a general slope of 0.25% is $95 \times 2.25 = 214$ cu. ft. per sec.

The accompanying tabulation indicates the degree of accuracy which can be obtained by use of the diagram. The discrepancies between the computed results and the results from the diagram are negligible.

Acres.	Slope.	$A^{\frac{4}{5}}\ S^{\frac{1}{2}}$ Computed.	Diagram.
5	2.0%	6.6	6.7
17	1.5	16.6	16.2
85	0.8	53.0	53.0
234	0.25	94.4	95.0
750	0.1	199.6	200.0

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It would be difficult to overestimate the importance of the conference to consider the conservation of natural resources which has been called by President Roosevelt to meet at Washington in May.

As most of our readers are aware, this conference is an outgrowth of the work of the Inland Waterways Commission. The members of this Commission, in their study of the broad question of the protection and regulation of the natural waterways, became impressed with the fact that this work was closely related to the broad question of preserving all our natural resources—streams, forests, farms and mines. The Commission, therefore, recommended to the President the issue of a call for a National Conference on this subject, and suggested that it should comprise the Governors of all the States and Territories, with delegates appointed by each, and representatives from leading organizations "engaged in dealing with natural resources."

It is not too much to say that the primary responsibility for this conference and for the whole movement to protect the gifts of Nature from waste and from monopolistic control lies with the engineering profession.

Engineers know far better than the men in any other calling how urgent is the need that something be done, and done speedily, to conserve our natural resources. As stated above, the original recommendation for a National Conference came from the Inland Waterways Commission, made up largely of engineers. Following this, the representatives of the National Engineering Societies on the Advisory Board for Tests of Fuels, petitioned the President to invite to this Conference representatives of the National Engineering Societies. In compliance with this request, formal invitations have been sent from the White House to the Presidents of these societies to attend the conference.

Further work is to be done by the National Engineering Societies, and we doubt not by other engineering associations, to arouse the interest of their members and enlighten the public on this important matter. Plans have been canvassed in New York City during the past week for the holding of a joint meeting of the four great national engineering societies, with speakers of national reputation to present the various aspects of forest preservation, stream regulation, prevention of waste in mining and in use of minerals, etc. If such a meeting is not found practicable, the several societies are likely to hold such meetings independently.

This is emphatically a matter in which united action by engineers through their professional organizations is imperative. Individual engineers the country over are deeply interested in this matter, as we well know; but the individual acting alone can accomplish little, while united action of engineers in their various societies the country over can have a powerful influence in awakening and instructing public opinion and compelling action on this most important matter.

We showed last week the influence which high prices for labor had exerted in bringing about the business depression. By high prices for labor, we do not mean high wages necessarily, for high wages often result in the most economical production. What we mean is paying out money with small net return in productive effect.

That the business depression has already brought about a large reduction in the cost of labor, there can be no doubt. In reducing forces there has been opportunity to weed out the inefficient and "tune up" the remaining workers to a higher standard of efficiency.

In addition to this, however, it will doubtless in many cases be necessary to put in force wage reductions, and where such reductions are made necessary by business conditions and are justified by conditions of supply and demand in the labor market, they are actually—paradoxical as it may seem—in the interest of labor as a whole.

It is an injurious fallacy that holding up prices by artificial means results to the general benefit of people as a whole. Mr. Samuel Gompers has publicly declared that if reductions in wages are made as a result of the business depression, strikes will be called. He has attempted to justify this on the plea that reduction of wages will reduce the purchasing power of the employees. The fallacy that lies at the bottom of this is the idea that the employer of labor has unlimited funds at his command with which to pay wages, so that the payment of high or low wages is a matter of his own personal preference. Actually, however, in any business where free competition prevails, the employer can pay only such wages as market conditions make possible. If he pays more, the margin of profit will be wiped out and his business will become bankrupt.

Mr. Gompers sees that if bricklayers, for example, are paid $5 a day instead of $3, they will have more money to spend; but he overlooks the fact that the extra $2 (assuming the day's work to be the same) is taken out of other people, who would spend the money just the same, so that the purchasing power of the community—the demand for food and clothing and shelter—would be the same as if the bricklayers earned $5 per day.

What Mr. Gompers overlooks further is that lowering of wages or of prices stimulates demand, increases the number of men that can be employed and as it lessens the number of men and machines that stand idle, it *adds* to the wealth-producing and purchasing power of the community. What would so stimulate the building industry, for example, at the present time, as a reduction of wages and increase in efficiency of labor, such as would encourage investors to undertake the construction of buildings on a large scale?

And what we have said of labor applies with equal force to materials. The Steel Corporation, for example, has the power to a large extent to maintain prices of its products. To do this doubtless largely increases its own revenues; but at the expense of a thousand other industries which have to pay a price out of proportion to the present relation between demand and supply.

Of course certain monopolized industries and their employees may thrive thus at the expense of the general welfare of industry as a whole; but some of the industries which have prospered in this way to a most extraordinary degree are finding nowadays that the public has correctly gaged the source of their prosperity and that at a time when they most want friends all voices unite in their condemnation.

Our leading article in this issue describes one of the most remarkable dams ever constructed in this country or, indeed, anywhere in the world—that across the Colorado River near the head of its delta at Laguna, a point twelve miles above Yuma, Ariz. The dam is, indeed, not yet completed, and some difficult work remains to be done before its final success will be assured. The coming campaign has been carefully planned, however, and there seems every reason to believe that the anticipated results will be secured.

The Laguna dam is unique among all American dam structures in that it is founded on shifting sand and silt. So much has been written in engineering literature during the past dozen years concerning the necessity of founding dams on rock, and on solid rock at that, and so many direful predictions have been made concerning a few dams which have not been carried to bedrock that it is refreshing to study the design of a dam which is built where a rock foundation or a hard pan foundation or an "indurated clay" foundation or any other foundation acceptable to the orthodox engineer was not to be had.

And yet the dam was needed and it was up to the engineers of the U. S. Reclamation Service to design it; and a year ago, when the contractor for the work "laid down on his job," it was up to these engineers to actually carry out their designs.

This dam has to be built, moreover, in the bed of a river which carries in its flood season over 100,000 cu. ft. of water per second; and when completed it must carry this flood safely over its crest. This crest, by the way, is very nearly a mile in length; and this is, of course, an essential feature for the safety of the structure. Were the flood flow concentrated in any short section of overflow, its velocity would be so great that undermining of the apron at the lower side of the dam and consequent destruction of the whole work would be the natural result.

The problem of making the final closure in a dam built in a soil which melts away like sugar when subject to a current of water is one of exceeding difficulty. This work has still to be done and we understand the plan now proposed is substantially the same that was adopted for closing the break in the Colorado below Yuma a year ago. A pile trestle is to be carried across the opening between the two ends of the dam and rock is to be dumped from this trestle all along its length. This will be done at low water, of course, when the flow of the river is reduced to 4,000 to 6,000 cu. ft. per second, and it is expected in this way to divert the river from the main channel through the bypass sluiceways that have been excavated through the granite rock at each end of the dam.

Behind the protection of a cofferdam thus built up, the engineers and their forces will work with all possible speed to close the gap in the main dam before a flood arrives.

The handling of the silt is one of the most difficult features of the whole work. The Colorado River waters carry in suspension past this point something like a million cubic yards a day of sand and mud, as an average throughout the year. So huge is the quantity of sediment that the reservoir formed above the dam, some ten miles in length and nearly eight square miles in area, will probably be filled with sediment as high as the dam crest within a year or so after the dam is completed.

Storage of water, therefore, is no part of the Laguna dam's function. It is simply a diver-

sion weir, whose purpose is to raise the water level high enough to permit the river to flow into the irrigating canals which start on each side of the river just above the dam.

If these irrigation canals led directly from the reservoir above the dam, they would inevitably be choked with sediment. Since the level of these canals must be lower than the crest of the dam, the sea of mud accumulated in the reservoir would tend to flow into the canals.

To provide so far as possible against this danger, the canals draw their water, not from the reservoir itself, but from the sluiceways at each end of the dam, through which the water will flow in a broad, deep stream, and the irrigation water will be drawn off near the surface where it carries the least sediment.

Notwithstanding the prevalent idea that railway traffic is at a standstill, it is evident when comparison with previous years is made that we are experiencing little more than a normal reaction. It is stated by the New York "Commercial" that at Indianapolis, in the week ending Feb. 8, 21,906 loaded freight cars were received and forwarded. For the same week in 1907 the figures were 22,183, and in 1906 the number of cars was 22,327.

TUNNELS FOR PASSENGER TRAFFIC AT NEW YORK CITY.

A quarter of a century ago, the completion of the New Croton Aqueduct tunnel, 40 miles long, for the water supply of New York City, furnished a great object lesson of the possibilities of tunneling by modern engineering methods. We recall, about the time of the completion of the Croton Aqueduct tunnel, a prophecy by some prominent man that the success of this great tunnel would lead to the undertaking of extensive tunnels beneath New York City, for traffic purposes. He foresaw a labyrinth of underground passages by which railway trains would pass underground at the outskirts of the city and would finally discharge their loads of freight or passengers near the points of their final destination.

We today are witnessing the fulfilment of this prophecy. Not, indeed in the exact form in which it was originally made; few prophecies are ever fulfilled to the letter. The distribution of freight underground has not yet been shown to be feasible in New York City or anywhere else, with the sole exception of Chicago's peculiar tunnel system. In urban passenger traffic, however, a very large proportion of the north and south travel on Manhattan Island is now carried by the subway; the first tunnel under the East River is in active operation; and there is opened this week with due ceremony the first tunnel line underneath the Hudson River connecting Manhattan Island with New Jersey.

Other tunnels under the East and North Rivers are likewise approaching completion. The Pennsylvania's two tubes under the North River were connected through some time ago, the first of its four tubes under the East River has just been joined, and on the other three the headings are near each other. The so-called "Belmont" tunnel under the East River is finished and nearly ready for traffic and the Cortlandt St. tunnels of the Hudson companies are far advanced. A few months hence, therefore, there will be six railway tunnels in operation underneath the Hudson River (or North River, as it is locally known in New York City) and eight tunnel lines underneath the East River.

It will be possible then for a passenger on either The Hudson Companies' system or the main line of the Pennsylvania R. R. to go underground at a point in New Jersey a mile or two back from the Hudson River, pass beneath that stream, go across Manhattan Island underground and beneath the East River and not emerge into daylight again until he reaches the outskirts of Brooklyn.

During the long years while the enemies of the New York Rapid Transit subway enterprise were preventing that enterprise from making a start, it was common enough to hear the prophecy that if the subway were ever built, people would refuse to ride upon it. "Time enough for people to go underground after they're dead," was the common sneer. Today the great problem of the subway managers is how to take care of the crush of traffic. Public opinion is unanimous as to the need of more subways. That they must and will be provided is accepted by all. The time of their construction will be determined solely by the financial ability of the city to raise the funds for the work.

Some features in the design of passenger traffic tunnels and their equipment are clear now that were little understood when the Rapid Transit subway was built. First of these we may note is the controlling influence of junction points on a line's traffic capacity. The number of trains per hour over a given track may be controlled by the number which can be run past a certain junction point. On the present Rapid Transit system, for example, the junction at 96th St. between the main line and the Lenox Ave. branch has been from the beginning a point of congestion. The more complicated the movement of trains over a system, the greater is the amount of interference at junction points and consequent traffic congestion and delays. For this reason the through train movements on any urban rapid transit system have to be limited. On The Hudson Companies' system of North River tunnels and connections, of which a map is given in this issue, there will be, when completed, at least five main terminal points. If it were attempted to run trains from each of these terminals to each of the other terminals, the interference and delays at junction points would become so great as to seriously cut down the traffic capacity and public usefulness of the system.

In the second place it has become evident that the traffic capacity of a line is closely governed by the length of station stops, and that this again is governed by the time required to load and unload passengers. On the present subway cars, with end doors, all the passengers desiring to leave at a station must get off before the crowd of waiting passengers can get on. The interference of the two crowds has made necessary the introduction of special policemen in the rush hours to keep the car doors clear.

All are agreed now that subway cars should have doors at both the sides and the ends so passengers can enter and leave simultaneously and the two streams may not interfere. Were the present subway cars equipped with side doors, it is probable that the length of stop at congested stations could be reduced to not more than one-third its present duration. The Public Service Commission of New York City has just ordered the Interborough Rapid Transit Co. to show cause why all its cars should not be equipped with side doors.

It is agreed also, that all cars for tunnel service should be of steel construction, and with incombustible materials throughout. The steel car is undoubtedly the coming car for general passenger service; but for tunnel service, where the possibilities of terrible casualty in case of fire are so great, steel cars are certain to be specified as an essential in the future.

It is interesting to recall that when the tunnels under the East and North Rivers now nearing completion were first projected, a great deal was said as to the great economy of tunnels as compared with bridges for these locations. There is reason to believe that the cost of the tunnels now approaching completion has much exceeded the estimates made when they were projected. It is probably true that a great bridge sufficient to carry, say, eight lines of railway track could be built across either of these rivers for less money than eight lines of single-track tube tunnel could be built underneath them. But the comparison does not end with the mere cost of the structure. The bridge requires much more expense in costly real estate for terminals than do the tunnels; and the bridge is a continuous source of heavy expense for maintenance, while the maintenance of a tunnel structure should cost hardly anything.

There have been some criticisms of the Pennsylvania R. R. officers for the enormous expenditures made in order to secure a terminal in Manhattan Island. Looking at the matter from a broad point of view, however, it is clear that some better means of passenger traffic movement across the river than the ferryboat had sooner or later, to be provided. Every year the number of craft plying upon the waters that encircle Manhattan Island increases, and with this increase comes delay and danger of collisions. The relief that will be afforded by the diversion of part of the trans-river passenger traffic to the tunnel trains beneath the river is a public benefit of no small proportions.

Further, as has been already pointed out in this journal, the terminal on Manhattan Island for main line passenger traffic is only one part of the Pennsylvania's large enterprise. Its terminal will also handle the enormous suburban traffic of Long Island, which now has to be handled by ferries. In the night hours when tracks can be spared, a large movement of freight will doubtless take place through the tunnel reaching the terminal tracks in Brooklyn, all parts of the Long Island R. R. system and, eventually, over the great bridge to be built over the East River to the main land, effecting through rail connection to New England.

MORE LIGHT ON THE WASHINGTON TYPHOID AND WATER SUPPLY PROBLEMS.

Those who believe that Washington, as the national capital, should set the national standard of municipal administration and public works have had no little cause for chagrin at its shortcomings as regards sanitary improvements. These shortcomings, we hasten to say, are mostly to be laid at the doors of Congress, rather than upon the administrative officers of the District. The efforts of those officers have been ably seconded by the Medical Society of the District of Columbia, which for many years past has conducted a vigorous campaign for sanitary reform, and particularly for pure water, safe milk and the reduction of the high typhoid death rate of the city. Congress has not been wholly deaf to the sanitary appeals made in behalf of the District of Columbia. In several particulars it has responded heartily, notably in making appropriations for the sedimentation and filtration of the water supply.

Certain events which have accompanied and followed the introduction of filtered water direct attention to other needed sanitary improvements at Washington. These events and needed improvements were set forth at length in our issues of Nov. 8, 1906, and July 11, 1907, and are again emphasized, in our current issue, in our report of an extended discussion of the water, milk and ice cream supplies of Washington in relation to typhoid fever in that city. Appended to our report of the discussion just named are some interesting and encouraging statistics, supplied at our request by the health officer of Washington, showing a decrease of 30% in the total number of deaths from typhoid fever in Washington in 1907, as compared with 1906.

Most of our readers are doubtless aware that Washington has been a heavy sufferer from typhoid fever for years past. A belief that water filtration would materially reduce the typhoid rate played a large part in securing the authority and money from Congress needed to build the slow sand filters that were put into full operation in October, 1905. Great was the disappointment when, after the filters had been in use for nearly a year, typhoid showed an increase rather than a decrease over the immediately preceding period. It was a natural offhand conclusion that the filters must be at fault, but the very low bacterial contents of the filtered water, and a study of the effects of filtration on typhoid fever in other cities cursed with typhoid, indicated that some other cause than the water supply must be responsible for the high typhoid rates in Washington.

It is easy to say that this might have been foreseen, especially since the cities whose typhoid had been cut down after introducing filtered water without exception had been using water heavily polluted with sewage at points relatively

near the water intakes, while the sewage pollution of the Potomac is introduced at remote points, and is subjected to great dilution before the water reaches consumers. But, although "residual" typhoid had been ably discussed by Professor Sedgwick, a few years before, the generally accepted belief of American sanitarians in 1905 and 1906 was that a continuous high typhoid mortality pointed unmistakably to polluted water, and demanded either filtration or a new supply. Unfortunately, the known coincidences of grossly polluted water supplies and high typhoid rates were so numerous as to obscure instances of fair and even unusually pure water and high typhoid.

As studies of the Washington typhoid proceed it becomes more and more evident that had not so much blame been put upon the water supply the real causes of the high typhoid in that city would have been found much earlier than they were, and that long before this most of them would probably have been greatly reduced. This, it seems to us, is forcibly shown by Dr. Rosenau's figures of known or believed typhoid origins for the years 1906 and 1907, given elsewhere in this issue. For 1906, 10% of the typhoid cases reported were attributed to the milk supply, against 7% in 1907. This decline may have been merely chance, but there is reason to believe that it was at least partly due to dairy inspection.

What seems to us of most significance is the increase of cases due to contact, or carelessness in the handling of patients, from 7% of all cases in 1906 to 19% in 1907, and the increase of imported cases (disease contracted outside Washington) from 15% in 1906 to 25% in 1907. If we are not greatly mistaken, these increases are apparent rather than real; that is, the percentages of contact and imported cases were fully as high in 1906 as in 1907, but they were not identified as such. This may be explained by the fact that in 1906 the investigations by Dr. Rosenau and his associates in the U. S. Marine Hospital and Public Health Service did not begin until the year was about half through, while in 1907 (we suppose) they not only covered the full year, but were presumably more efficient than in 1906, due to the increased experience of the investigators. The same reasoning, it may be noted, leads to the conclusion that more than 10% of the cases of 1906 may have been due to milk, and that dairy inspection in 1907 may, therefore, have done more good than the apparent reduction from 10% to 7% would indicate.

Both the total number of cases and the total number of deaths from typhoid fell off materially in 1907 as compared with 1906. The summer rise was notable, as is usual in Washington and also in other places where high typhoid rates are attributable to other causes than water. In Washington, the 1907 summer counts of bacteria in the filtered water were low at the very time when the typhoid was high, while high bacterial counts in December were neither accompanied nor followed by high typhoid.

The cause for the decline of typhoid in Washington in 1907 has not yet been fully and clearly explained. The dairy inspection and the falling off in milk cases account for only a small part of it. An interesting subject for speculation, and one well worth careful local study, is the heavy decline in typhoid among the colored, as compared with the white population. How can this be accounted for? The first explanation that suggests itself is improved conditions of one kind or another that tended to reduce the *actual* number of contact cases. It seems fair to assume that if the larger number of contact cases recorded in 1907, as compared with 1906, was chiefly due to the greater disclosure of such cases, rather than to any actual increase, then the preventive measures against the spread of the disease must have been more active and efficient in the later than in the earlier year, and thus the total number of cases may have been reduced thereby.

Even if a better knowledge of local conditions should indicate that the foregoing speculations are baseless, we think that our reflections on the subject, combined with the indisputable facts of the case, point unmistakably to the emphatic need of searching investigations into the probable cause of every case of typhoid fever that occurs in any city of the country.

Striking as are some of the facts and possibilities which we have outlined, they are scarcely as significant as others brought out last week at the Washington meeting. The revelations as to the unsanitary conditions prevailing at the dairy farms supplying the District of Columbia, as to the contamination of the almost sterile filtered water by dirty ice and dirty water coolers, and as to the filthy condition of ice cream manufactories, were enough to shock anyone save the most hardened sanitary inspector.

It is little short of marvelous that anyone should have been so ready to place on the filter plant the blame for the continued typhoid fever at Washington, during 1906, when that plant was practically the only scientifically designed, constructed and operated barrier to the spread of typhoid in the city and when its efficiency was being measured and certified every day by standard methods of analysis. Meanwhile, a dozen other avenues of infection were either inadequately guarded or not guarded at all, and this is not the worst; for, whereas typhoid fever is the chief danger from polluted water, milk and other foods may be the vehicles of infection by tuberculosis, scarlet fever, diphtheria and other communicable diseases.

Before concluding, we feel compelled to temper our direct and implied commendation of the Medical Society of the District of Columbia by a few words of criticism of its attitude, or that of at least some of its members, towards the use of coagulants as an occasional aid to treating the Potomac River water at times of very high turbidity. Preference to slow sand rather than mechanical filtration can be understood, especially a half dozen years back, but there seems to be no valid reason for a determined stand against the occasional use of a coagulant when the water is very turbid, merely as an aid to the preliminary clarification effected by the large storage reservoirs. Figures given elsewhere in this issue show that there are always far more than enough carbonates in the river water to neutralize all the sulphate of alumina that could be advantageously used when the turbidity is at the highest.

Every engineer connected with the development and operation of the Washington filtration plant has urged the importance of using a coagulant at times. By that means, both turbidity and color could be reduced, dangers of clogging the filters and upsetting their bacterial efficiency would be obviated, and the expenses of operation could be cut down.

Dr. Rosenau (not speaking for the Medical Society, however) expressed himself as preferring additional storage, with its resulting sedimentation, to coagulation. He had no objection to alum, but thought that the well-known general improvement incident to the storage of water would be highly advantageous, besides which, additional storage would prolong the period during which the river could be excluded from the aqueduct during stages of high turbidity. The advantages of storage and sedimentation are recognized by all well informed persons, but so much of both have already been provided at Washington that it is certainly questionable whether more should be added unless such a plan would be less expensive than occasional coagulation. Dr. Rosenau stated that coagulation would be the less costly of the two. The fact is, that the additional capital outlay needed to make coagulation possible would be insignificant compared with that for additional storage, and the yearly outlay for coagulants would be small compared with the interest charges on another storage reservoir.

If the turbidity problem can be solved by any economically feasible increase in storage facilities, and if plentiful appropriations for this and other needed sanitary work in and for the District of Columbia can be secured, certainly more storage would be the proper plan; but if, as we surmise, District appropriations come hard, and coagulation would be less costly than more storage, ought not a coagulant to be adopted, and the money thus gained utilized for increasing the frequency and efficiency of dairy and other sanitary inspection and for equipping and maintaining a bacterial laboratory? Then, too, unless the U. S. Public Health and Marine-Hospital Service is to continue to serve in that capacity, Washington needs to be provided with a high-grade typhoid detective bureau, so that in the future it need not, as in the past, work so blindly and sometimes so laggardly in combatting its typhoid scourge. The need for such detective work is forcibly illustrated by the fact that prior to 1906 comparatively little evidence was collected which could properly be used to place the blame for the high typhoid on any specific deficiency in the sanitary works or administration of the city, while more thorough investigations begun late in 1906 placed 32% of the cases on milk, contact, and infection outside the city, and the full year of more experienced work in 1907 increased the accounted-for cases to 51%. We believe still more can be accomplished by continuing and improving the same general line of work, but even the results thus far achieved cannot be equaled unless the Health Department or some branch of the general government be provided with ample funds and well-trained men to carry on the work.

LETTERS TO THE EDITOR.

The Temperature of the Gases of Combustion in Experiments on Burning Milwaukee Garbage.

Sir: The article on "Garbage and Refuse Disposal at Milwaukee," in your issue of Jan. 16, 1908, contains the words:

> . . . the heat required for heating the form of combustion gases to the temperature of 1,500° F., which they possess when they pass out of the chimney, etc.

I do not understand this statement, as I have understood the temperature of flue gases in the chimney to be about 500° F.

Very truly yours,
W. M. Booth.

Dillaye Building, Syracuse, N. Y.,

[Professor Sommer, from whose report Mr. Booth quotes, has responded as follows to our request for a reply to the foregoing query.—Ed.]

Sir: I am sorry to say that the sentence in question states something I had not intended to say. Instead of the words, "pass out," it should read, "pass into." Hence the sentence should read:

> Practically this calculation should be somewhat modified by adding the heat required for heating the formed ashes and the heat required for heating the formed combustion-gases to the temperature of 1,500° F. which they possess when they pass into the chimney.

Mr. Hering asked me to make a chemical analysis of the garbage and to determine its calorific value, in order to find out whether the garbage will burn itself without the addition of coal. But the calorific determinations as made with our scientific instruments (Mahler's bomb, Parr's calorimeter, etc.) give only the most favorable, ideal results, no heat being permitted to be carried away by the superheated water-steam, by the formed carbon dioxide and by the other gaseous substances formed by the combustion.

In a practical furnace for incinerating garbage, such ideal conditions do not exist, and when it is desired to calculate the efficiency of such a furnace in terms of energy it actually received from the combustion of the garbage, in order to find out if this energy is sufficient to cause the evaporation of the water contained therein, the only logical procedure is to determine, for any given furnace, the temperature at which the products are passing away just after they have ceased to do work (the evaporation of the water) and calculate the quantity of heat contained in them for subtraction from the ideal calorific values, which the chemist obtains with his ideal instruments. The last paragraph in my report refers to this matter.

Very respectfully yours,
R. E. W. Sommer,
Professor of Chemistry, Marquette University
Milwaukee, Wis, Jan. 27, 1908.

[Professor Sommer's assumption of 1,500° F. is correct for the purpose of his discussion, which really deals with the nominal or assumed temperature in a garbage furnace. If such a furnace was connected directly to the chimney the gases would then enter the chimney at that temperature, but if the heat was utilized in a

boiler the temperature at the base of the stock might then be 500° F., as Mr. Booth suggests.

In some British refuse destructor installations the heat passing the boiler is still further utilized. Thus at Watford, England (see Eng. News, April 20, 1905, pp. 406-7), there is first a regenerator for heating air for the forced draft and then an economizer for heating the boiler feed-water both between the boiler and the chimney. At the time of the guarantee test of this destructor, to quote from the article named:

> The maximum temperature in the combustion chamber of the destructor, which is back of the grates, was reported as 2,700° F. and the "average approximate temperature" as 2,000°. The temperature of the gases in the boiler downtake was 1,500° maximum; 800° minimum; 1,100° average. At the chimney base the temperatures were 455°, 300° and 400°, maximum, minimum and average.

During the test the by-pass damper controlling the passage of the gases of combustion to the economizer was open only half way, since the station regulations do not permit a feed-water temperature, at the economizer outlet, in excess of 300° F. Under this condition, it is interesting to note, the feed-water entered the economizer at 53° F. and left it at an average of 243.2 and a maximum of 322° F.—Ed.]

A Defence of the Elastic Theory of Arch Analysis, and a Caution.

Sir: In your issue of Jan. 9, 1908, Mr. Daniel B. Luten says of the elastic theory:

> I do not believe that any method of applying this theory to masonry arches can be trusted to produce a design at once safe and efficient. There is no question in my mind that the elastic theory is correct; the entire trouble is with the assumptions on which it is based and the difficulty of application.

Let me try to meet some of Mr. Luten's objections that follow, one of which is that the end tangents are not fixed because the abutment is itself elastic and must change shape. In fact, even if we go to the foundation; that too is elastic, so it is impossible for the end tangents at abutment or foundation to remain fixed in direction. This is true, but the answer is, that with sufficient abutment and a good foundation, carrying a sufficiently low unit-stress, nearly uniformly distributed, the change in direction of tangents is practically negligible. This is true because the resistance to bending of the abutment (or foundation) is so much greater than that of the thin upper part of the arch whose elastic deformation mainly determines the curve of the centers of pressure. In my book "Theory of Steel-Concrete Arches," 4th ed., p. 41, it is stated:

> If the abutment should be included as part of the arch, its influence on the pressure would be found to be almost nil, which is in fact the reason why it is treated as inelastic and immovable.

Where the abutment is very small, then it should certainly be included as a part of the arch. It adds but little to the labor of the computation to do so. In this case its neutral line must be found by use of the same principles as used for the arch.

Mr. Luten next finds fault with the results for temperature stresses. Unfortunately the formulas from which they are found are true, on neglecting the uniform compression and making the usual approximations. In the book cited above, pp. 69-75, three formulas for temperature stresses were compared and gave results differing only from 2 to 4%. Two of these formulas involved approximations relative to the arch ring; but the third, the well known formula of Winkler, Greene, Howe and others, was found by integration and did not involve errors incidental to the graphical methods. And yet they all gave practically the same results.

If steel-concrete arches suffered the extremes of temperature of steel arches, then we should have to stop building them without hinges; but fortunately they do not. Earth and concrete are very non-conducting substances. Prof. Howe states, in "Symmetrical Masonry Arches," p. 119, that

> a recording thermometer placed in the ring of a reinforced-concrete arch, having earth filling, indicated that the total range of temperature change did not exceed about 20° F. in some ten or twelve months.

If similar observations should be taken all over the country, we should soon have data enough to settle upon the extremes of temperature to allow for.

As to Mr. Luten's remarks about cracks in continuous spandrel walls contradicting theory, I am unable to follow him. I have always found, in the many pressure lines drawn, the movement of the arch to correspond exactly to the proper pressure line.

It is well known that as a train moves over a bridge, the stresses change on many joints from near zero to tension, to a maximum compression or the reverse, so that it makes a stronger arch to have reinforcement near both extrados and intrados, though it is not absolutely necessary everywhere. Theory does not call for a double reinforcement throughout. Personally, I should admit a little tension, for extreme loading, in an arch ring. What reason can be urged that it should be allowed in a steel-concrete beam and not in an arch?

As to the axial stress being ordinarily omitted, I am sure a good designer would include it in a very flat arch.

As to the earth filling causing active earth pressures, as computed by usual formulas, after the earth has been thoroughly tamped and cohesive and chemical affinities have come into play, I am something of a skeptic. I believe the pressures are small and diminish with time.

The backing, by its passive resistance to a spreading of the arch, adds to its stability.

What has given me more concern than any of the objections named, results from the nature of concrete as regards its imperfect elasticity, variable permanent set and shrinkage in air. The latter is unknown and it is hoped that some experimenter will measure it on straight beams, both of concrete and reinforced concrete, to 0.0001-in. When the shrinkage of the span can be computed, it replaces the corresponding term in the formula for horizontal thrust due to temperature changes. Large stresses may result from this shrinkage, which at present cannot be computed, because the coefficient of shrinkage is not accurately known.

In a reinforced arch, the metal resists this shrinkage, thus setting up internal stresses, compression in the steel, tension in the concrete, exactly as in a reinforced beam. In a plain concrete arch, shrinkage has full play. Until more is known on this matter, it seems unwise to project large reinforced concrete arches of several hundred feet span; for any minute change of span, whether due to the giving of the abutments, temperature changes or shrinkage of concrete, causes correspondingly large stresses.

I have thus been careful to point out candidly what may prove to be weak joints in our armor; for a good constructor must consider every influence for good or evil in his bridge. It does not follow, because there is some doubt on some points or hypotheses, that therefore we should discard all theory and design by rule of thumb. We can just as logically discard all theory in connection with reinforced-concrete beams, particularly restrained ones, because the hypotheses are approximate.

The elastic theory for arches has proved of great service in design; particularly in eliminating undesirable forms and in reducing sections at the right points and increasing them at others. It is the only rational theory we have and it has come to stay. Our object now should be, as far as possible, to eliminate some uncertainties that are due to lack of sufficient observation of a new material, including temperature ranges, shrinkage, internal stress and sets under repeated loadings.

Respectfully,

Wm. Cain.

Chapel Hill, N. C., Jan. 16, 1908.

The Lacing of Compression Members.

Sir: There have appeared recently in your correspondence columns several letters commenting on the writer's article, "The Lacing of Compression Members," which appeared in your issue of Nov. 7, 1907.

The theory, as stated in my letter to you at the time, was submitted as a suggestion which might prove useful to designers until experiments can be made and the actual stresses more nearly determined. I am very glad that it has stimulated so much interest and discussion and I will give below my reasons for the various assumptions made.

The stress was assumed to be uniform over the cross-section of each leaf because the leaves, between lattice connections, are not supported in any way, and if the lattice connections are at the neutral axis of the leaf, the deflection of the leaf between lattice connections would be just as liable to occur in one direction as the other, and thus it might be the inner or outer fiber of the leaf which would receive the maximum stress, in fact the probabilities are that the deflection would alternate in direction in successive lattice panels, but this deflection is not due to the bending of the column as a whole, but to the direct compression in the leaf itself. The two leaves act as the two chords of a truss and not as the two flanges of a girder with a solid web.

For this reason it seems to the writer that the assumption of uniform distribution of stress over the area of the leaf is justified.

Mr. Henry S. Prichard questions the assumption that the shear increment is uniform from end to end of the column. I grant immediately that the shear increment is not uniform, but the maximum increment is not $\pi/2$ times the average for any practical column length or load, and while it is true that the moment varies as the sine or rather the cosine curve, the increment can never attain the value of $\pi/2$ times the average except for the ideal long column, nearly concentrically loaded and at the point of failure. The proof of this statement is given as follows:

Let L = total length of the column.
l = the length from a point of contraflexure to the nearest point of maximum bending.
$= \frac{L}{2}$ for a pin ended column.
$= \frac{L}{4}$ for a fixed ended column.
P = total load.
e = eccentricity of the load, due either to initial curvature, accidental or intentional eccentricities.
x and y = coordinates of the elastic curve as shown in the figure.
E = modulus of elasticity.
I = moment of inertia.
A = total area of cross-section.
r = radius of gyration.

The diagram shows only the part of the column covered by the length l. A complete discussion of this phase of the subject is given in "Stresses in Structures," by Prof. A. H. Heller, Chapter X.

The equation of the elastic line is found by integration to be

$$y = e \sec \theta l \operatorname{vers} \theta x \quad \ldots\ldots(1)$$

in which

$$\theta = \sqrt{\frac{P}{E I}} \quad \ldots\ldots(2)$$

The bending moment at any section is

$$M_x = P (e + y_{max.} - y) = Pe \sec. \theta l \cos \theta x \quad \ldots\ldots(3)$$

Differentiating with respect to x we get the shear at any section,

$$\frac{dM}{dx} = S_x = -Pe\,\theta \sec. \theta l \sin \theta x \quad \ldots(4)$$

and differentiating the shear we get the shear increment per unit of length,

$$\frac{dS}{dx} = v = -Pe\,\theta^2 \sec \theta l \cos \theta x \quad \ldots\ldots(5)$$

The increment is a maximum when $x = 0$ and is,

$$v_{max.} = -Pe\,\theta^2 \sec \theta l \quad \ldots\ldots(6)$$

The total shear to be transmitted is given by equation (4) when $x = l$ and is

$$S_{max.} = -Pe\,\theta \tan \theta l \quad \ldots\ldots(7)$$

The average shear increment then is

$$v_a = \frac{S_{max}}{l} = -\frac{Pe\,\theta}{l} \tan \theta l \quad \ldots\ldots(8)$$

This corresponds to the value of v used by the writer in his original article.

By combining equation (6) and (8) we get

$$v_{max.} = v_a \frac{\theta l}{\sin \theta l} \quad \ldots\ldots(9)$$

The maximum value of θl will *not* be $\frac{\pi}{2}$ for a practical column, as assumed by Mr. Prichard, but considerably below it.

From equation (2)

$$\theta l = l \sqrt{\frac{P}{E I}} = \frac{l}{r} \sqrt{\frac{P}{A E}} = \frac{l}{r} \sqrt{\frac{s_c}{E}}$$

Assuming the straight line formula before used

$$s_c = 16{,}000 - 70 \frac{L}{r} = \frac{P}{A}$$

and values, for the constants which will make θl as large as possible as follows: $E = 30{,}000{,}000$; $\frac{L}{r} = 150$; $l = \frac{L}{2}$ (pin ends), we get, $\theta l = 1.0155$ and substituting in equation (9) we get, $v_{max.} = v \frac{\theta l}{\sin \theta l} = 1.19\ v_a$ instead of $1.57\ v_a$.

For the actual column the partial fixity of the ends and accidental inequalities in material and workmanship will render l less than $\frac{L}{2}$ and seldom is a column used as slender as that given above.

In developing the formula in the original article the writer wished to avoid the use of a formula which would be as difficult of application as equation (9) above, and it was for this reason that the average unit stress was used instead of the maximum, *after* the writer had made the above investigation of the effect of the assumption.

There are so many conditions affecting the actual column in practice, such as initial stresses and bends, non-homogeneity of material and accidental eccentricities of loading, that, in the opinion of the writer, the above assumption is justifiable when it is taken in connection with the other assumption that $l = \frac{L}{4}$, which is the case in the ideal column with fixed ends and is approached by *all* practical columns, especially when the pin is parallel to the plane of the lacing *as is usually the case.*

When a column is intentionally eccentrically loaded, Eq. (6) above may be used in proportioning the lacing, but it will usually be found to give a less shear increment than Eq. (1) of the original article because P will be less.

There is no more warrant for the assumption that the moment curve is a parabola by Mr Prichard than that it is a straight line as assumed by the writer in his original article. Both assumptions give results approximately correct with the advantage on the side of the straight line because by the assumption of the parabola $v_{max.}$ is made *double* the average instead of less than 1.2 times the average.

Thanking you for allowing this lengthy discussion, I am,

Yours very truly,

Clyde T. Morris.

Ohio State University, Jan. 11, 1908.

[The question at issue between Mr. Prichard and Prof. Morris may be made somewhat clearer by looking at it in another way.

If a built column be subjected to direct compression by two equal and opposite forces applied to its ends, it bends outward slightly. The internal forces or stresses acting at any cross-section of the member are (1) a direct compression, which is parallel to the axis of the column at that point and therefore inclined to the line of the external forces, and (2) a shear, transverse to the axis. As there are no transverse external forces, the transverse components of the internal stresses must balance. Therefore the shear is equal to the transverse component of the longitudinal stress. The latter, as a sketch, will readily show, is substantially equal to the product of the load by the tangent of the angle between the column axis at the section considered and the line joining the centers of the ends. Expressing this in briefer form: If the shear be called S, the load P, and the slope of the column axis dy/dx, $S = P\frac{dy}{dx}$. This shear is greatest at the ends of a pin-ended column, or at the points of inflection in a continuous column, or speaking more generally, at the point where the slope of the deflected column axis is greatest. The shears at the various sections are represented graphically by the degrees of slope of the column axis at the respective sections. The mean slope between center and end of column represents the average shear. A line drawn from the middle of the column to the center of the end would have a slope equal to the average shear, while a tangent to the column axis at the end represents by its slope the maximum shear (in a pin-ended column). The ratio of the two slopes is the quantity in question. What values might be assigned to this ratio in practical use can readily be judged by the individual designer. It is not wholly a matter for algebraic analysis, since the deflected column axis may show an unusually steep inclination on a short section, due to initial crookedness or other irregularity, and the shear would be correspondingly large at this section. It is safest in all cases to take the maximum shear at a larger figure than the average value deduced by Prof. Morris; how much larger is more properly to be settled by judgment than by analytics.—Ed.]

Riveted Lattice or Pin-Connection for Railroad Bridges: An Answer to the Lattice Argument.

Sir: The writer notes in your issue of Jan. 23, 1908, an article on "Riveted Lattice Bridges of Maximum Span," by Geo. H. Thomson, in which, as the writer looks at it, some very erroneous ideas are brought out.

Bridge engineers may apparently be divided into three classes: First, those who favor pin connections for all purposes. Second, those who favor riveted construction for all spans. Third, those who try to determine the relative merits of each, and in each particular case adopt one or the other for specific clean-cut reasons based on its adaptability for the purpose in hand.

To make the writer's position clear he will state frankly that he has adopted riveted construction for one bridge over the Mississippi River of 266 ft. span, for highway purposes; a 200-ft. span over the Red Lake River, at Crookston, Minn., and a similar one crossing the Red Cedar River, at Menomonie, Wis.; and for the Ferry bridge at Duluth, Minn., over the ship canal, nearly 400 ft. in span. On the other hand he has used partial pin-connection for a heavy railroad bridge over tracks for the Minneapolis, St. Paul and Sault Ste. Marie Ry. of but 160 ft. in span. In each case, however, the same general line of reasoning determined the type adopted.

Let us consider the claims of lattice versus pin-connected structures from the standpoint of safety; first, structures for railroad use. The following facts in the development of bridge design should be borne in mind:

1. Numerous failures have occurred in inferior designs of both classes of bridges. Assuming that the same degree of skill had been used in the design of each type. Since the pin-connected type chiefly has been used in the United States, and this country early possessed about two-thirds of the mileage of railway track in the world, it should be expected that there would be more than double the number of pin-connected bridge failures than riveted failures.

On referring to the old files of Engineering News, we find a failure recorded in Switzerland, of a bridge whose plans bore the name of Eiffel, a riveted lattice structure. We find failures of both types in the United States. We find cases under derailment where members of the single-intersection pin-connected type have been bent considerably out of line, and the structure still remained intact. An example of this came to the writer's notice only recently in crossing a pin-connected bridge over the Mississippi River. Looking out of the car window, he noticed a hip vertical of eyebars bent several inches out of line (through a blow perhaps of some freight train). We may question, perhaps, whether proof has been furnished that the single-intersection type, with its larger and heavier members, is more subject to destruction in case of derailment than the double-intersection with its lighter members.

However, the advantage of the pin-connected type from the standpoint of economy for structures of spans up to 300 ft., carrying relatively light loads, may be open to question. Comparisons have generally been made between the pin-connected design with economical proportions and the riveted type of uneconomical proportions of depth and panel length. The writer's experience indicates that, given an economical diagram for both types, for the lighter loads there is little difference in their cost. The riveted structure requires less work in the shop, somewhat more field riveting and some additional weight as compared with the pin-connected type, and these several items nearly balance each other in most cases.

The reason of the writer's preference for the riveted type for lighter structures is found in its greater stiffness in the joints, obviating counters and adjustable members except in lateral rods; in the feasibility of better lateral connections in the riveted type, and in the somewhat greater stiffness of the structure as a whole. These considerations are sufficient, in the writer's judgment, for the adoption of the all-riveted type for lighter work.

When we come to heavier construction the following considerations present themselves. First, what test data do we possess as to the strength of large riveted tension members? Second, what do we know about the degree in which the various parts of a riveted built-up tension member act together in resisting applied loads? Third, what test data have we as to the exact condition of the rivets in a joint involving a large number of rivets distributed over a large gusset plate.

Mr. Thomson, in his article, suggests that we get close to nature; use only so many of her laws as we can truly decipher by the aid of mechanical tests truly applied. His own advice would bar us from the use of built-up tension members of large dimensions, because we have absolutely no test data whatever showing the strength of large built-up members, while on the other hand we have many full-size tests of large eyebars carried to destruction, and the deportment of such bars is well known.

The objection which Mr. Thomson urges against eyebars, regarding the molecular distortion of the eye, is equally applicable to the smaller pin in the shape of a rivet. [Except for the effect of plate friction.—Ed.] As to standing aghast at the fact that large eyebars show some little yielding under the pressure of the pin on the head, what shall we say about the similar yielding in the riveted plate-girder under load, a phenomenon which has been known and observed (see Rankine) by the earliest writers? Are we to stand aghast and condemn all structures, or are we to select the best and most economic and least objectionable details which have been used heretofore?

The failure of the great Quebec cantilever seems to have provided material for the prejudiced advocate of riveted construction to decry American methods of construction in general, and to assume that the pin-connected structure is unfitted for practical use, although the records of the best and most conservative designs of this type have neither been equalled nor excelled by riveted construction. Compare the deterioration from rust which takes place in riveted tension members, with that which occurs in the heavy eyebar tension member; with the latter there is much less area exposed, and therefore much less deterioration. As far as concerns damage by range of stress, the miles of eyebar chords in this country which have been doing business for twenty years and more under higher working stresses than they were designed to carry is a fair answer to criticism.

Mr. Thomson states in the conclusion of his article that, in the light of the greatest bridge failure known, he presents again his plea for rational construction along lines of which we have absolute knowledge. If Mr. Thomson possesses this absolute knowledge of the strength of large built-up tension members, it hardly seems fair to the rest of the profession that he should keep the information all to himself. The rest of his brethren interested in bridge building are ignorant of the data. Moreover, they know of no machine in existence today capable of furnishing such data.

As we examine the photographs of the wreck of the greatest bridge failure, and observe in this wreck the riveted members reduced to scrap, and broken, bent, distorted and twisted in all kinds of shapes; and at the same time see the eyebar chains, which have fallen for the most part from a greater height, practically intact and little damaged, are we profiting by the light of this great bridge failure if our respect for heavy eyebar chains be not increased ten-fold? When we consider that the failure of this great structure was caused by defective riveted members, not by a failure of a pin-connection nor by the eyebar chains, are we profiting as we should by this failure if we fail to recognize in it a lesson for the designing of either type of construction? A lesson in the makeup of riveted members, more than anything else, except teaching that the heavy eyebar can stand more abuse and hard usage than any other tension member which the engineer is able to devise!

In comparing the Forth Bridge with the Quebec design we may question whether the tension members of the Forth Bridge, if tested to destruction, would develop the factor of safety they are assumed to possess, just as the compression members of the Quebec cantilever failed to develop the strength they were assumed to possess, when the load actually came upon them. The difference between the two cases lies partly in the fact that the engineers of the Forth Bridge employed more conservative working stresses for their structure than did the designers of the Quebec Bridge, and it may be possible that, should these tension members ever be called upon to carry as high a stress as it was proposed to place on the eyebars of the Quebec cantilever, advocates of riveted construction may have cause for reflection, just as the profession at large finds much to learn in the failure of the great undertaking at Quebec.

The fact that through defective design of part of the work this failure did occur should not lead to the condemnation of those features of the construction which it would be difficult, indeed, to improve upon. Take, for example, the diagram of the Quebec cantilever; in general proportions it would be difficult to improve upon it, though from the artistic standpoint the design is unquestionably open to criticism. It has something of a top-heavy appearance, and it gives a general impression of weakness at the anchorage which, as the writer looks at it, is unwarranted from the standpoint of cost and might be readily remedied. Similar comment would hardly apply to the Forth Bridge as built.

Among advocates of riveted construction it may be noticed that most misleading formulas for proportioning members have been devised, and their influence, indeed, has been most unfortunate on the design of American types of bridges. The Germans have originated the maximum-and-minimum formulas of proportion, which are irrational as applied to practical work. An Englishman is the author of the dynamic theory, which, though much more conservative, is in logic almost equally as objectionable.

In conclusion, it cannot fail to occur to the thoughtful engineer that the lesson of the Quebec Bridge failure is not in any wise a condemnation of the pin type of construction, but rather a lesson of equal force in both types of construction. Hence the absurdity of the extreme advocate of riveted construction holding up this failure of the Quebec cantilever as an unanswerable reason why pin-connected structures are inherently dangerous and ill-fitted for practical purposes, especially in the field where they possess their greatest merit and inherent advantages over the riveted type.

Mr. Thomson speaks of erecting 2,000-ft. spans on the lattice principle without risk of erection. Let us quote Mr. Benjamin Baker, speaking at Montreal in reference to the Forth Bridge when the work was under way:

> If I were to pretend that the design and building of the Forth Bridge was not a source of present and future anxiety to all concerned, no engineer of experience would believe me. Where no precedent exists the successful engineer is he who makes the fewest mistakes.

This modest statement of this eminent engineer does not agree with the complacent assertions of the extreme advocate of riveted construction. Mr. Baker indicates clearly his appreciation of the exceptional difficulties in erection of the Forth Bridge. Considering the means at their command and the state of the art at that time, the skill exhibited by the constructors of that bridge can but command our highest admiration. Yet when we count the cost and speed of erection with

that shown practicable by the work done on the Quebec cantilever, the advantages of the pin-connected type are strongly emphasized, and this experience is not changed or rendered less valuable by the fact that a serious mistake had unfortunately been made in the general makeup of the riveted portions and in the adoption of absurd fatigue formulas, formulas which have originated among the strongest advocates of the all-riveted type of construction.

Taking up now specifically the question as to the slight distortion in the eye of the bar due to the clearance between the eye and the pin, we are brought face to face with the question as to whether a minute permanent set in a portion of the metal is or is not a dangerous proposition. Take, for instance, the shell of the boiler; in the rolling of the plate a permanent set in compression on the one side and in tension on the other is given the shell in the course of manufacture. There are none in the business that consider this slight deformation as in any wise unfitting the metal for safe practical use. This distortion of the plate is much greater than that in the eyebar.

In the matter of the forging of the head: If the metal is so seriously damaged by forging, and for that reason so universally condemned by the English engineer, why should he use forged crank-shafts and use forgings in nearly all parts of his machine design? Such parts are subjected to repetition and range of stress from tension to compression, a kind of usage that has proved most destructive to specimens under test where range of stress has exceeded well defined limits.

Is not Mr. Thomson drawing rather upon prejudice than upon clean-cut, well defined proof for his antipathy to the eyebar?

Let us consider some of the examples he presents for the lattice bridges. Take, for example, Fig. 5, the lattice bridge marked "Hilton, 1865." One may question whether a more objectionable arrangement of the metal in either a tension or a compression member has ever been devised. In fact, is it not on a par with that in the compression members of the Quebec design? Its inherent weakness would unquestionably be developed by severe tests, or even under low working stresses if given an economic panel length.

Engineers of the younger school of to-day are not seriously impressed by any one's mere opinions; they demand proof. They believe that the distribution of stress in a tension or compression member is capable of fairly exact measurement by the application of methods and apparatus familiar to all students of physics. Before Mr. Thomson can legitimately expect this class of his professional brethren to display very enthusiastic approbation of his views, he must needs apply these methods and furnish proof as to the uniformity of the distribution of strain over and through cross-sections of his riveted construction. He must show this school of professional men beyond question that he has facts and not mere opinions to present for their consideration.

Among the statements that are hardly impressive to the young engineer is the assertion, by those who favor the suspension type of bridge, that the strength of the cable is determined when we know the strength of a specimen of a single wire of the bundle. We are quite in ignorance as to how perfectly the several wires will work together in the cable and how perfectly they will be adjusted so that they are in a position to work together. The same question we naturally raise in the matter of the riveted tension member made up of a large number of pieces. In the large eyebar or group of bars connected by a pin with fairly accurate shop work we have less doubt as to the degree of uniformity of the distribution of strain over the full sectional area.

These are the questions in which the advocate of large built-up tension members must furnish proof by actual measurement, before he can expect his views to be accepted without question.

C. A. P. Turner.

Phoenix Bldg., Minneapolis, Minn., Jan. 29, 1908.

Notes and Queries.

The total storage capacity afforded on the Croton drainage area, which forms a part of the water supply of New York City, should have been given as 104,530,000,000 gals., on page 155 of our issue of Feb. 6. As careful readers of the article must have observed, three ciphers were inadvertently omitted.

ELECTRIC MOTOR DRIVES have been adopted for the mills of the Clark Thread Co., situated on both sides of the Passaic River at Newark, N. J. It is proposed to gradually replace the isolated driving plants now in operation by a large central power station in which there will be five turbine units aggregating 10,000 HP. capacity. The development of this plant and of the whole electric drive scheme will proceed as circumstances allow the present system to be superseded without interference. At first a 2,500-KVA., 600-volt, three-phase, 60-cycle, Allis-Chalmers turbine unit will be installed. About a third of the output of this unit will supply induction motors in the cotton, spooling, finishing and packing mills on the east side of the river and the remainder will be transmitted under the river to two cotton mills and to a thread mill on the west side. The plans for this development have been made by Mr. J. H. Abercrombie, M. Am. Soc. M. E., Mechanical Superintendent of the Clark Thread Co. It is stated that much of this work is based on the experience of J. & P. Coates, Ltd., of Paisley, Scotland, the largest thread makers in the world, with which the Clark Company is affiliated.

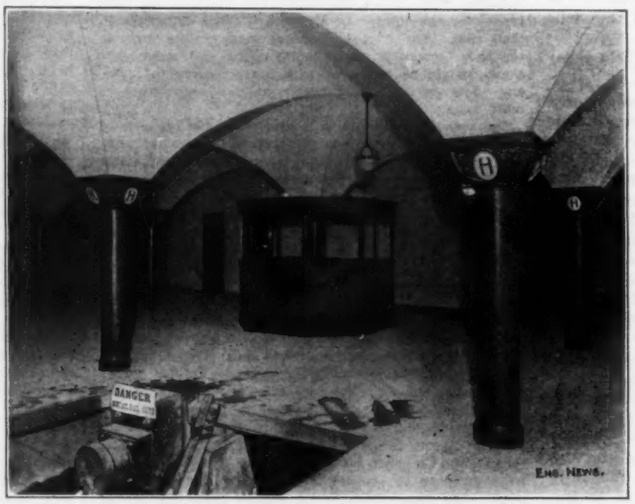
A 5,000-VOLT SINGLE-PHASE RAILWAY of meter-gage in Canton Tessin, southeastern Switzerland, was opened for traffic a few months ago, between Locarno (on Lake Maggiore) and Bignasco, along the valley of the Maggia River. The line is about 17 mi. long and has grades up to 3½% and curves as sharp as 250 ft. radius. A water-power generating station at the Locarno end supplies the power for operation. Oerlikon side-contact collectors are used on the cars. Each car has four 60-HP. motors for 200 volts each.

THE OPENING OF THE FIRST HUDSON RIVER TUNNEL.

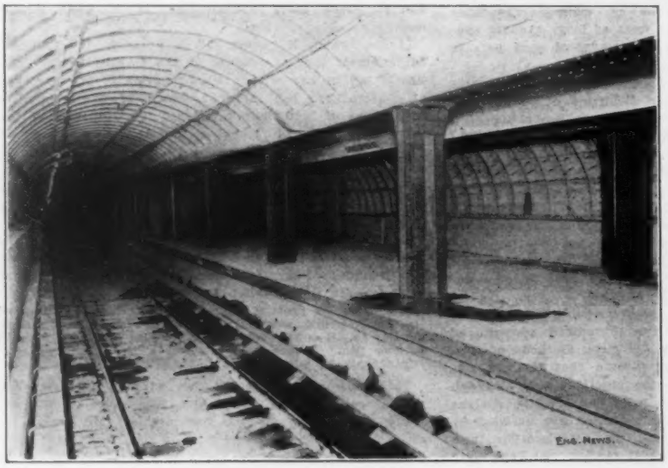
Tuesday, Feb. 25, 1908, saw the official opening of a great work, the first tunnel under the Hudson River, after a history of discouragements and delays that has few parallels in modern construction. Almost exactly one-third of a century has passed since the work was started. In November, 1874, a man more bold than experienced, De Witt C. Haskin, started the excavation of the New Jersey shaft for a tunnel under the Hudson River, at a time when such a tunnel was almost universally regarded as an impossibility. His plans and methods proved inadequate to the task. Constructive difficulties and financial anemia paralyzed the enterprise, though it was taken in hand again and again. Not until a quarter century later was the proper conjunction of greater skill, modern methods and ample money brought to bear on the work with success. In 1902 Charles M. Jacobs took charge of the work as Chief Engineer, for the New York & Jersey R. R., and since then the advance has been uninterrupted to the present stage of completion.

A glance at Fig. 2, a map of the system of the Hudson & Manhattan R. R. Co. (the present name of the operating owner), indicates that the system of to-day is far larger and more perfect than that planned in 1874. But of this larger system, comprising 9 mi. of double-track line, only about one-third, 2.85 mi., is yet ready for operation, and this is the 1874 tunnel under the Hudson together with about 1½ mi. of shore approaches. The opening ceremonies of Feb. 25 therefore are essentially in celebration of the long-delayed success of the First Hudson River Tunnel.

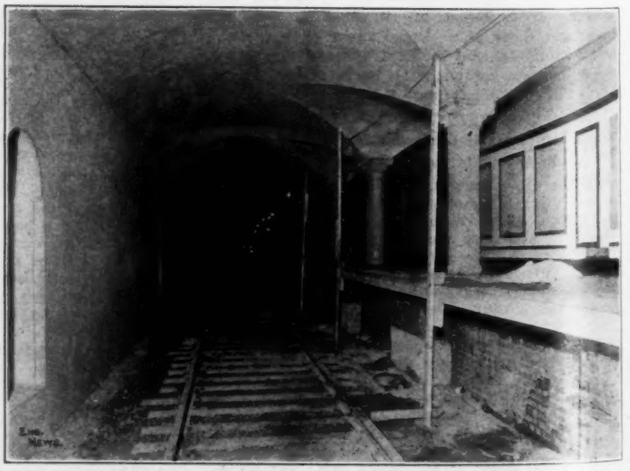
It will be fitting to take a brief survey over the checkered history of this old work before we speak of the present system. Our pages have reported this history in much detail, so much so that a mere list of the references



PORTION OF TRACK FLOOR OF HOBOKEN TERMINAL STATION.



CHRISTOPHER ST. STATION PLATFORM.
(Central Platform Between Two Iron-Lined Tunnels.)



NORTH BOUND PLATFORM OF FOURTEENTH ST. STATION, LOOKING NORTH.
(Platform on outer side of track at all stations in the reinforced-concrete section. Wall at left is the dividing wall between northbound and southbound track compartments.)

FIG. 1. INTERIOR VIEWS OF THREE STATIONS OF THE HUDSON AND MANHATTAN R. R. CO.

would occupy several columns. From these reports the following historical summary has been gleaned.

Historical Summary.

FIRST PERIOD: D. C. HASKIN'S WORK.

The Hudson River Tunnel Co. was incorporated on May 23, 1873, to build a tunnel under the Hudson River between New York and Jersey City, for establishing communication with the railways terminating in Jersey City. De Witt C. Haskin was president of the company. The plan was to build a tunnel 26 ft. wide by 24 ft. high, large enough for two tracks for regular railway cars and locomotives. The tunnel was to have a boiler-iron shell lined with 3 ft. of brickwork. Borings and preliminary work occupied a year.

November, 1874. Excavation began for a circular shaft 30 ft. in diameter at the foot of 15th St., Jersey City, about 100 ft. from the seawall. The shaft was sunk 20 ft. when the Delaware, Lackawanna & Western Ry. secured an injunction against continuance of the work. This and other suits stopped all work for five years.

September, 1879. Litigation over, work was resumed. The shaft was completed to 60 ft. depth about the end of November, and lateral penetration of the tunnel began.

December, 1879. Air pressure first put on the tunnel excavation on Dec. 28. The use of compressed air to support the face and hold back the water, *without a shield*, was the method by which Mr. Haskin expected to put through the whole tunnel. About this time it was decided to substitute two single-track tunnels of elliptical shape, about 16 ft. wide by 18 ft. high for the large double-track tunnel, with only 2 ft. of brick lining. The engineers under Haskin at this time were Spielman & Brush.

February, 1880. The first plate of the permanent part of the North tunnel was put in place Feb. 9.

July, 1880. Toward the end of July, the North tunnel was 280 ft. out, the South tunnel 15 ft. A bad blowout occurred on July 21, during the work of enlarging the junction just at the shaft, drowning some twenty men. Prior to this the tunnel work had gotten much out of line and grade and cracks had developed in the brick lining. The work of making an entry into the tunnel from a pneumatic caisson sunk alongside the shaft took all the rest of the year.

January, 1881. Work was resumed where the accident had interrupted it. A 6-ft. pilot tunnel was now used, this method of advance being introduced by J. F. Anderson (later of Anderson & Barr), Superintendent of Construction (only until May). In May, 1881, Gen'l. Wm. Sooy Smith took charge as Chief Engineer, but he remained only about a year.

August, 1881. A caisson was started at the New York end, foot of Morton St.

December, 1881. At the New Jersey end, the North tunnel was advanced to about 680 ft., the South tunnel to about 525 ft.; the rate of progress was about 3 ft. per day, with air pressure 22 lbs. Some advance was accomplished without excavation, by forcing back the face after plastering it with balls of worked silt.

March, 1882. The New York caisson was completed. A bad settlement occurred in the New Jersey North tunnel. Two airlocked bulkheads had been built in the New Jersey North tunnel, with 32 lbs. pressure at face of heading. The top part of the lining was now made 2½ ft. thick.

July, 1882. The New Jersey end progressed about 4 ft. per day. The North tunnel was about 1,100 ft. out. S. H. Finch was Engineer-in-Charge at this time.

November, 1882. Work was stopped for lack of funds. The following was the state of advance at this time:

New Jersey North tunnel	1,542 ft.
" " South "	570 ft.
New York North tunnel	74 ft.

December, 1882. Trenor W. Park, President of the company, to whose faith in the enterprise much of the financial support had been due, died at sea. C. G. Francklyn succeeded him.

April, 1883. Work was resumed at the New York end. By August the North tunnel had been advanced to 150 ft., the South tunnel to 20 ft.

August, 1883. All work was again suspended, "awaiting new financial arrangements."

March, 1886. The tunnel was pumped out for examination.

April, 1887. Work was resumed at the New Jersey end.

September, 1887. Work was again suspended. The New Jersey North tunnel now was 1,840 ft. out from the shaft.

SECOND PERIOD: PEARSON & SON'S WORK.

March, 1888. The tunnels were again pumped out. Sir J. Fowler and Benjamin Baker became Consulting Engineers, but did not assume personal direction or responsibility.

March, 1889. Mortgage bonds were floated in England, and work began soon afterward, with S. Pearson & Son of London as contractors. Wm. R. Hutton was Engineer-in-Charge, and E. W. Moir was Superintendent for the Contractors. By the end of 1889 the New Jersey North tunnel had been advanced to about 2,025 ft., the other headings not being worked. Early in 1890 preparations were under way for putting in a shield about 2,025 ft. out. A bad blowout occurred in the course of this work.

About May the shield was completed, and the advance of the North tunnel with regular pneumatic shield working began, a 2½-in. cast-iron lining being put in behind the shield. The shield was much smaller than the old tunnel in outside dimensions, being about 19 ft. in external diameter. The cast-iron lining segments were 20 ins. long, and their flanges 8 ins. high; the outer diameter was 18 ft. 9¼ ins., the inner 17 ft. 5¼ ins. The first part of the lining was too weak, and later a stronger lining was designed and used. Some of the early lining cracked.

January, 1891. Regular progress in New Jersey North tunnel was at the rate of 7 ft. per day; the shield was working well. By April the length of the New Jersey North tunnel had been increased to 3,400 ft., the rate of advance now being 10 ft. per day. By May the advance was to 3,700 ft. This was already past the State Line, which is just beyond 3,000 ft., but as the deepest channel is on the New York side, the bottom of the grade was not reached until July, 1891.

On account of a rock reef some 600 ft. long projecting above the grade of the tunnel invert not far ahead, difficulty in holding the air was anticipated, and permission of the War Department was sought to place a blanket of clay on the river bottom. This was obtained in June or July, but probably never acted on.

August, 1891. All work was suspended at short notice, owing to lack of funds. The stringency in the London money market resulting from the Baring failure made it practically impossible to secure more money. The New Jersey North tunnel had been advanced to 3,916 ft. at

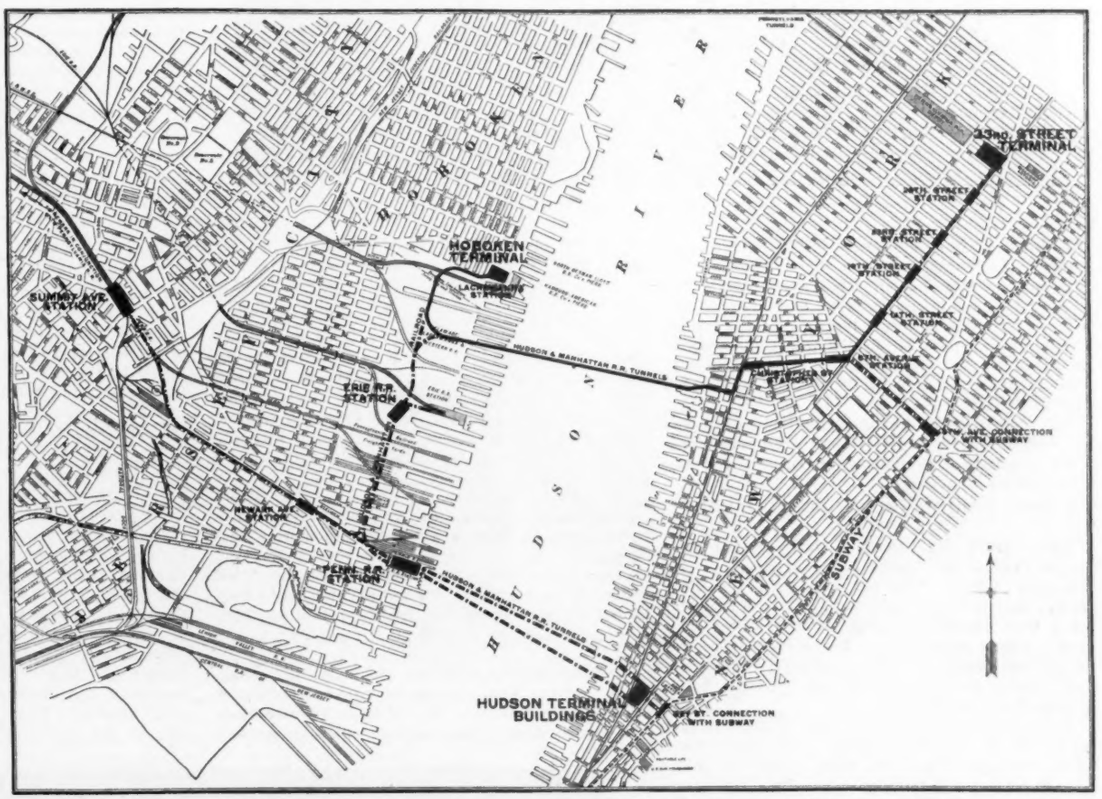


FIG. 2. MAP OF HUDSON RAPID TRANSIT TUNNEL SYSTEM CONNECTING NEW YORK WITH JERSEY CITY AND HOBOKEN; HUDSON AND MANHATTAN R. R. CO.

The part shown by solid line was opened for traffic on Feb. 25, 1908. The line is doubletrack throughout. The subaqueous tunnel at Morton St. comprises two singletrack tunnels side by side. The subaqueous tunnels at Fulton St. and Cortlandt St. are each singletrack. The turnout of the Ninth St. branch at Sixth Ave. and 9th St., New York City, is at grade. The crossings at the two junctions in Jersey City are not at grade, the junctions being doubledeck.

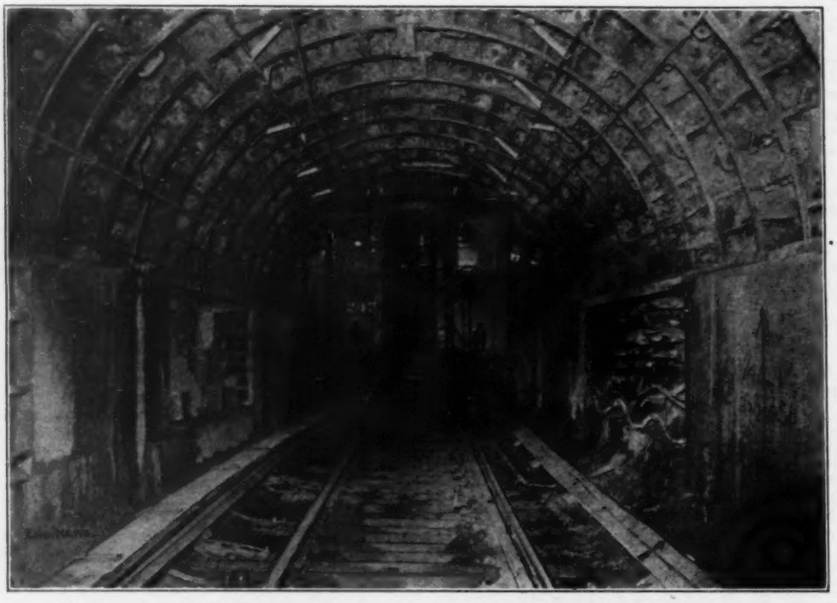


FIG. 3. VIEW LOOKING EAST IN WESTBOUND TUNNEL, IRON-LINED SECTION.

the time work stopped. It rested here for almost 11 years.

FORECLOSURE AND REORGANIZATION.

April, 1892. S. Pearson & Son filed a mechanics' lien for about $26,000 against the tunnel.

1896-97. Charles M. Jacobs was commissioned to report on the tunnel to the bondholders, and presented an estimate of the cost of completing the work.

1899. Sale of the tunnel under foreclosure was ordered in March, and on June 16 the tunnel was sold to Stetson, Jennings & Russell for the bondholders.

THIRD PERIOD: CHARLES M. JACOBS' WORK.

1902. Work was resumed early in the year under Charles M. Jacobs as Chief Engineer, by company labor. The old shield in the New Jersey North tunnel was used, with the addition of a 6-ft. horizontal apron on the center line, and a shield built for the South tunnel; no attempt was made to advance the New York headings, which had been closed off by brick bulkheads at an earlier period.

The financial or administrative history of this period is not known in detail. The New York & Jersey R. R. began the work, under Mr. Wm. G. McAdoo as President. In 1903 this was merged with the Hudson & Manhattan R. R. Co., organized to build two tunnels from a point under the Pennsylvania R. R. station at Jersey City to Cortlandt and Fulton Sts., New York. Still later, in 1905, a corporation was formed under the name The Hudson Companies to conduct the construction and real estate operations for the Hudson & Manhattan R. R. Co. Under this arrangement the enterprise still remains.

March, 1904. The North tunnel was holed through on March 11.

The South tunnel was then pushed ahead rapidly, much use being made of the method of advancing by closing the shield doors and shoving ahead without excavation. Phenomenal rates of progress were recorded with this method, as much as 62 ft. being done in 24 hrs., and 348 ft. in a single week!

The rock reef near the New York shore was the chief difficulty. Here clay was dumped into the river over the tunnels as a blanket, and successful use was also made of the scheme of heating the silt face of the heading with torches to dry it out and bake it, in order to enable drilling and blasting to go on in the rock projecting up into the lower part of the section. The apron proved an invaluable aid here.

The South tunnel was made smaller in diameter than the North tunnel, as its size was not fixed by any existing shield. The South tunnel is 15 ft. 3 ins. inside the lining and 16 ft. 7 ins. outside, the flanges being 8 ins. high.

The old portions of the tunnel, which were very irregular in line and grade, were "faired" as much as necessary by cutting and adding lining concrete. The west end of the South tunnel, which had drooped to a 6% grade in the old work, was cut through from the new portion on a flat slope, so as to reach a level entirely below the North tunnel in the approach section. This made it possible to avoid crossing at grade at the junction Y with the north-south line in Jersey City (see map, Fig. 2).

The New York City work was done by shield tunneling to Twelfth St. and Sixth Ave. The part north of this is a reinforced-concrete section.

September, 1905. The South tunnel was holed through on Sept. 30. Its entire length of about 5,100 ft. was excavated in 334 actual working days, an average of 15.3 lin. ft. per day.

This condensed summary chronicles the struggles and the final success of an undertaking of remarkable difficulty. The Hudson River bottom is composed of a fine mud or silt, soft enough when permeated with water to flow through small openings, and so unstable as to offer but precarious support to a tunnel. The water is about 60 ft. in maximum depth, and on most of the width of the river the silt bottom is very deep. Near the New York shore a reef of rock projects up to near the river bottom, and any tunnel at the site selected must pass partly through this rock; the silt overlies the rock directly, so that this part of the tunnel must be bored with its lower edge in rock and its upper edge in the treacherous silt, a condition never before encountered in subaqueous tunnel work.

It is quite clear to-day that the original scheme of this enterprise was visionary, impossible, both as regards the construction of the tunnel and as regards its utilization. On the former subject nothing need be said further than that the experiences obtained during the prosecution of the brick-lined work proved the impracticability of the method used. Shield tunneling and cast-iron lining were necessary to ensure success. This refers as much to the matter of maintaining line and grade and avoiding cracking of the structure as to the possibility of putting the tunnel through at all.

Concerning the utilization of the tunnel, it is to be remarked that the plan of the original promoters of the Hudson River tunnel was to run the trains of the steam railways terminating in Jersey City and Hoboken through the tunnel to New York. So far as we know, none of the railways ever entered into the project; its practicability is highly questionable. Electric traction was not known at that time; compressed-air and banked-fire locomotives were spoken of for the service. In its recent stages the enterprise has never contemplated anything but electric traction, the success of this being established in the late 80's for trolley operation, and in the 90's for third-rail train operation. The present system of course employs third-rail traction exactly similar to that used on other urban rapid-transit railways.

The enlargement of the original Morton Street tunnel enterprise by the addition of the Fulton-Cortlandt Street tunnels and the two extensions (1) northward into the heart of New York City by a subway through Sixth Ave. to 33d St., and (2) westward from Jersey City to a connection with the tracks of the Pennsylvania R. R. for direct operation to Newark, eight miles west of the Hudson, has vastly increased the importance of the enterprise to the transportation system of New York City. At the same time it complicated and magnified the engineering problems of the work very greatly. These latter cannot be entered into at the present time. Suffice it to say that

FIG. 4. CROSS-OVER WEST OF GREENWICH AVE. STATION, LOOKING EAST.

the subaqueous work whose progress was recounted above is not the only section which involved great difficulties, nor can it with assurance be set down as the section of *greatest* difficulty. Portions of the land tunneling, in fact, required more original methods and more delicate execution than any of the subaqueous tunneling.

The relation of the complete tunnel system to New York City may be followed out readily by reference to the map Fig. 2. A description of the system is first necessary.

The southerly pair of tunnels, or lower tunnels, taps New York City about half a mile north of the Battery, the southerly end of Manhattan Island. A pair of 22-story office-buildings has been erected over the terminal loop, on the westerly side of Church St., covering the two blocks between Cortlandt St. and Fulton St. This is within five minutes walk of the Subtreasury, the center of the financial district. The New York Rapid Transit Subway, on Broadway, one block east of the Church St. terminal, will be brought into direct connection with it by an underground foot passage along Dey St., entering the south end of the Fulton St. station of the Subway. From the Church St. terminal westward to the river there will be four tracks, two under Fulton St. and two under Cortlandt St. These connect with two single-track tunnels which cross the river, converging in a station lying directly under the Jersey City terminal of the Pennsylvania R. R. Thence the line continues double-tracked in a west and a north branch, each double track. The former rises to the surface and enters upon the main line of the Pennsylvania R. R. just east of the cut through Bergen Hill. The north branch extends along Washington St., Jersey City, in tunnel, about 1¼ mi. north, crossing under the tracks of the Erie Ry., to a junction with the west approach of the upper tunnels. At both of the junctions mentioned, the opposing tracks separate to two superimposed levels, thereby avoiding crossings at grade. From the upper junction the line continues north, crossing under a river slip and then under the tracks of the Delaware, Lackawanna & Western Ry. Just north of the tracks it turns east to a large underground terminal station alongside the Lackawanna trainshed and ferry-house.

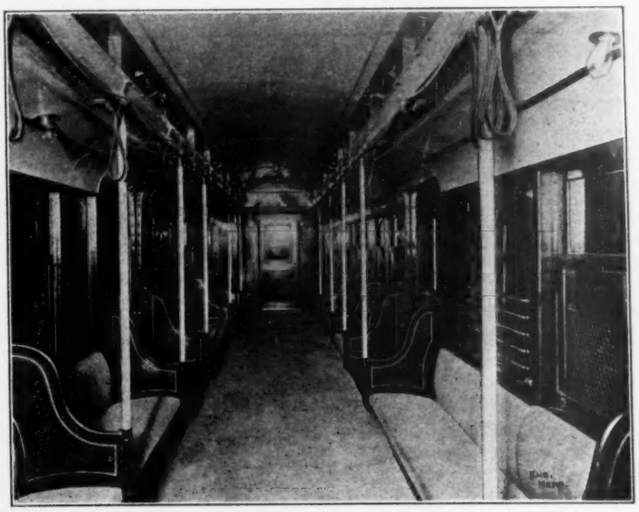


FIG. 5. INTERIOR OF CAR, HUDSON AND MANHATTAN R. R.

The historic upper tunnels extend from the junction eastward under Fifteenth St., Jersey City, with a maximum down grade of 5%, under the river, to a depth of about 90 ft. below low water, then rising with a 3% maximum grade to the old caisson at the foot of Morton St., New York. Excluding the old brick portion of the subaqueous tunnels, all this work is iron-lined tunnel, the two tracks being in separate circular tunnels, except that the Hoboken junction is of concrete, constructed in a caisson of reinforced concrete sunk from the surface.

The New York approach rises rather steeply eastward along Morton St. to Greenwich St., thence north on Greenwich St. two blocks to Christopher, and turns east on Christopher St. At Christopher and Greenwich Sts. is the first station.

The original plan of the New York & Jersey R. R. Co., when it took up the work of completing the old tunnel, was to use the tunnel for carrying the trolley cars of the North Jersey network from Hoboken and Jersey City to New York, bringing them to a subsurface terminal loop at Christopher and Greenwich Sts. On this account the New York end of the tunnel was sloped upward more steeply than is required by the larger scheme developed later. However, the grade serves to bring the Christopher St. station near enough to the surface to make elevators unnecessary.

The line extends east along Christopher St., still as a pair of circular iron-lined tunnels driven by shield, to Sixth Ave., where Sixth Ave., Christopher St., Ninth St. and Greenwich Ave. intersect. Here the Greenwich Ave. station is located. The grade along Christopher St. is flat, being determined by the requirement of the old Rapid Transit Commission that at Hudson St. (a block east of Greenwich St.) and at Greenwich Ave. the tunnels be kept so low as to give at least 20 ft. clear below the street surface for future rapid-transit subway construction.

At Sixth Ave. the line turns north, and also branches eastward into Ninth St. The main line extends up Sixth Ave. to 33d St. The shield-driven tunnels continue to Twelfth St., rising steadily. Here begins a reinforced-concrete subway type of construction, and continues the remainder of the distance to the 33d St. terminal.

The tunnels under the river have variable spacing, averaging about 30 ft. c. to c. for the upper tunnels. The land portions of the iron-lined work have a spacing of 18 ft. to 25 ft., depending on local conditions.

The iron-lined tunnels generally have an outer diameter of 16 ft. 7 ins. and an inner diameter of 15 ft. 3 ins. inside the iron (8-in. flanges). The upper river tunnels have different diameters, as already noted. On curves the standard dimension is changed to 16 ft. 6 ins. outside and 15 ft. 4 ins. inside, with 7-in. flanges. The sharpest curves, at Morton St. and at Greenwich Ave., are of 150-ft. radius on the center line of the inner track. The iron-lined tunnels have an inner facing of concrete over the invert and up to the center line (Fig. 3). The cable ducts are embedded in concrete benches on either side.

The reinforced-concrete section consists of two rectangular single-track compartments separated by a 15-in. dividing wall. The dimensions of each compartment are 13 ft. wide by 14 ft. 6 ins. high inside; the inner line of the roof is arched, and the height is that at center of compartment. The cable ducts extend along the outer wall, and the track therefore is placed 6 ins. inward of the center line, making the spacing of the two tracks 13 ft. 3 ins. c. to c. Where the columns of the Sixth Ave. elevated railway rest on the structure, the roof is strengthened by transverse steel I-beams. The grades of this part of the line were determined by the piping, sewers, etc., encountered below the surface.

The reinforced-concrete section is being constructed by the Degnon Contracting Co. All the rest of the work is done by the Hudson Companies directly.

This tunnel system provides the first means of crossing the Hudson at New York City otherwise than by ferry. Its southerly end is in the center of the financial and office district of the city, while the northerly end extends through the department store district and terminates at a very central point; it is close to the new city terminal of the Pennsylvania R. R., now building; it is just at a station of the Sixth Ave. elevated railway, and it is within a few steps of Broadway and close to the theater district. The Ninth Ave. elevated railway is crossed at Christopher St., where both elevated and tunnel line have a station. The Ninth St. branch will terminate at the Astor Place station of the subway, which extends the full length of Manhattan Island and into the Bronx, and through the Battery tunnel under the East River to Brooklyn.

The New Jersey portion of the tunnels connects with the Erie, Lackawanna and Pennsylvania railway terminals, having stations directly alongside or under each of them. It gives equally direct connection with the trolley lines of Jersey City and Hoboken, which radiate thence outward to all of northern New Jersey. The westward branch of the southerly tunnels will give train service directly to Newark, via the tracks of the Pennsylvania R. R., and will relieve the latter of a heavy burden of short-haul traffic between New York and Newark.

The utility of the system may be better understood, perhaps, by noting that the portion opened this week carries a passenger from 14th St. to the Lackawanna terminal in but eight minutes, for which trip the previously available methods of transportation required at least twenty minutes to half an hour. The fare in the tunnels is 5 cts. between any two points, against a fare of 8 cts. when using surface car and ferry. With the opening of the rest of the system the comparison will become far more favorable to the tunnel route. The operating equipment of the tunnels consists of all-steel cars operated by direct-current third-rail, with multiple-unit control. Power is supplied from the generating station in the southern part of Jersey City, at 11,000 volts, three-phase, converted to 600-volt direct-current at three substations. The present system is fed from a substation at Christopher and Greenwich Sts., which building also contains the entrances to the Christopher St. station.

The cars will be run in trains of eight during the rush hours. They embody several important new features. There are both end and center doors, to facilitate traffic interchange. The doors are arranged to give an electric signal to the motorman when all doors of a train have been fully closed, and the bell-cord method of signaling for the start is thus done away with. The cars contain no cross-seats, but only side seats, and these are continuous, i. e., not divided by arms limiting the number seated. Standing passengers are provided not only with the well-known straps, but also with steel uprights set at intervals along the front edge of the seat rows. Fig. 5 shows the interior of one of the cars.

The stations of the entire system are so located as to avoid curved station platforms with the inevitable wide gap between station and car platforms. The terminal stations have separate platforms for arriving and departing passengers, to prevent the extra crowding produced by the crossing of two lines of movement. The 33d St. and the Hoboken terminals are stub terminals, while the Church St. terminal will be a double-track loop. The operating portion of the system has intermediate stations at Christopher and Greenwich Sts., Sixth Ave. and Greenwich Ave., and along Sixth Ave. at 14th and 19th Sts. Stations on the unopened part of the Sixth Ave. line are provided at 23d and 28th Sts., besides the 33d St. terminal.

Fig. 1 shows three views of typical stations on the portion opened for traffic this week. The view in the Hoboken terminal represents only a small portion. This is a large station, with two floors below ground, the upper for communica-

tion, ticket booths, etc., the lower for the train platforms. Both floor and roof are formed in groined arches of concrete.

The Christopher St. station, also shown, is the only one except the Greenwich Ave. station to have a center platform. This station was constructed in a novel way. In the case of the London tube railways, room is provided for stations by substituting a length of large tube for the regular tunnel. Here, however, the space between the two tunnels was utilized. The two separate tunnels were driven and completed past this point, and then a small segment-shaped roof-shield was started in the space between the two iron tubes, resting on the outside walls of the tubes near the upper quarter. A cast-iron segmental lining was put in place behind this shield as it advanced, and then the soil below was excavated down to near the level of the tunnel inverts. Foundations for columns to support the roof being prepared on this bottom, the exposed side plates of the tunnels were removed, longitudinal steel girders were placed under the junction of tunnel wall and intermediate segment roof on both sides, and columns set to support these girders. A concrete station platform was then built up to the desired level, and a concrete roof arch was filled in below the segmental roof plates down to the bottom line of the longitudinal girders.

The stations in the reinforced-concrete section are of the type represented by the third view in Fig. 1, showing the north-bound side of the 14th St. station. The dividing wall between the two compartments is not broken, and the two station platforms are located on the outer sides of the respective tracks.

The complete separation of the two tunnels, maintained by the dividing wall in the reinforced-concrete section and interrupted only at Christopher St. station, the Greenwich Ave. station, and two-cross-overs located just west of the latter (Fig. 4) and at 19th St., is relied upon to take care of all needs in the way of ventilation. The train movement in either compartment will act, like a series of pistons moving in the same direction, to produce a steady forward current of air. Additional ventilation will be furnished by fans at several points of the system forcing fresh air into the tunnels and sweeping the heated and vitiated air out of the station openings.

The Hudson & Manhattan R. R. Co., which operates the system, is under the presidency of Mr. William G. McAdoo. Mr. Walter G. Oakman is President of the Hudson Companies. Mr. Pliny Fisk and Mr. Wm. M. Barnum of the banking house of Harvey Fisk & Sons, have been closely connected with the financing which made the work possible. The engineering and construction work was done by Mr. Charles M. Jacobs, M. Am. Soc. C. E., Chief Engineer, and Mr. J. Vipond Davies, M. Am. Soc. C. E., Deputy Chief Engineer. The total cost of the tunnel system and its equipment will reach $70,000,000.

WILHELM HILDENBRAND.

Wilhelm Hildenbrand, M. Am. Soc. C. E., one of the foremost engineers of America in the field of suspension bridge design, died in New York City on Friday, Feb. 21, after a brief attack of pneumonia. Mr. Hildenbrand was in his sixty-third year. He was born on June 1, 1845, at Karlsruhe, in the Grand Duchy of Baden, Germany. He received a classical education in the Lyceum of his native town, after which he entered the Polytechnic School of Karslruhe. Here he studied engineering under Redtenbacher, noted as one of the founders of scientific mechanical engineering, and under Sternberg, one of the originators of scientific bridge construction based on the principles of applied mechanics. These two men had made the Polytechnic School of Karlsruhe the most renowned in Germany and many foreigners, including men from Norway, Russia and Brazil, were among the students.

After graduating in engineering and passing a strict examination, according to the rules of the State, Hildenbrand entered the employment of the Government, and served one year as engineer of highway construction and as inspector of rails at the rolling mills of Westphalia and Rhenish Prussia.

In 1867 Hildenbrand emigrated to America. His first position was as draftsman in an architect's office, but he soon found employment with John A. Roebling, who was at that time planning the construction of the Brooklyn Bridge. Under Roebling's directions Hildenbrand made the first drawings of that famous structure, in the building of which he was afterwards to take a prominent part.

It took some years for financiering the Brooklyn Bridge project and removing the legal and other obstructions before its actual construction was put under way. During this time Hildenbrand found employment as an assistant engineer with the New York & Harlem R. R. The time was propitious. Vanderbilt had just made arrangements to unite the New York & Harlem, New York Central and New York, New Haven & Hartford railways in one grand terminal passenger station at Fourth Ave. and 42d St. in New York City. Hildenbrand was entrusted with the architectural design for the building as well

W. Hildenbrand

as the construction of the great arched roof over the train shed. Both structures were erected with strict observance of his plans and specifications, laying the foundation of his reputation as an architect and engineer. The roof truss had at that time the largest span in the world and has endured to the present time.

In 1870, when the Grand Central station was far advanced towards completion, the actual construction of the Brooklyn Bridge began and Hildenbrand returned to the service of Washington A. Roebling, who had succeeded his father, John A., as Chief Engineer of the Brooklyn Bridge, John A. Roebling having died in 1869. Hildenbrand became the principal assistant to Roebling, and had to carry most of the responsibilities of the construction during Roebling's severe and protracted illness which confined him to his bed or room for nearly ten years. Hildenbrand made all the scientific investigations and mathematical calculations necessary for the structure, and he also made the architectural design for the approaches and had charge of the steel superstructure, which, under his direction, was designed, inspected and erected.

After the completion of the Brooklyn Bridge, in May, 1883, Hildenbrand opened an office for practice as a civil engineer and had charge of the building of numerous bridges in different parts of the country, principally suspension bridges, but also a truss bridge over the Ohio River at Wheeling, W. Va., which has been given much praise for its aesthetic design. In 1885, Hildenbrand submitted a design, in public competition for the Washington bridge over the Harlem River and was awarded the second prize. The bridge as it now exists conforms closely to his design, the first prize plan having had entirely different dimensions and proportions, though in general appearance it was similar to Hildenbrand's plan.

In 1894 Hildenbrand did important work for the Chamber of Commerce of New York City, which had protested against the building of a pier in the center of the North River for a railroad bridge which at that time was projected by the New York & New Jersey Bridge Co. The President of the United States appointed a commission of five distinguished engineers to examine the feasibility of a single-span bridge across the river. Though four of them were at the outset disposed to favor a cantilever bridge with a river pier, Hildenbrand's demonstrations and plan submitted for a single-span wire suspension bridge convinced them of the superiority of such a design and they recommended unanimously a single-span wire suspension bridge, substantially on the lines of Hildenbrand's plan, and condemned the pier as unnecessary and obstructive to navigation. The report of this commission put a final end to all projects for placing a bridge pier in the North River.

In 1895 Hildenbrand became Chief Engineer of the reconstruction of the Covington & Cincinnati bridge over the Ohio River, and this is his most important independent work. The bridge ranks second only to the Brooklyn Bridge, and is now the third largest suspension bridge in the world. This bridge was originally built by John A. Roebling in 1867, but had become inadequate for the increased traffic. Hildenbrand replaced the entire structure, except the original stone towers, with a larger, stronger and more substantial suspension bridge, and did the task without interrupting the travel on the bridge for a single hour. The bridge was finished in 1899 and has been in service ever since, to the full satisfaction of the owners and an ornamental landmark to the cities of Cincinnati and Covington.

Immediately on completion of the Covington bridge, Mr. Hildenbrand went to Mexico, where he constructed another suspension bridge, of a span equal to that of the Cincinnati bridge, but of smaller width and lighter construction, its purpose being to transport silver ore from the mines of the Peñoles Co. at Mapimi to the terminal point of a rack railway, built on the "Abt" system, which Hildenbrand had constructed in 1897.

Roman Abt, of Lucerne, Switzerland, the inventor of the important system known by his name, for ascending with locomotives and trains steep inclines up to 25%, made Hildenbrand his representative for America. In this capacity Hildenbrand had constructed, previous to the Mapimi rack road, in 1889 and 1890 a similar road to the summit of Pike's Peak, which was and still is the highest railway in the world, reaching an elevation of 14,214 ft. above sea level.

In 1900 Hildenbrand entered the service of the John A. Roebling's Sons Co., of Trenton and New York, as Chief Engineer for the cables of the Williamsburg Bridge, for building which the above-named firm had the contract. These cables are now the largest in the world, exceeding those of the Brooklyn Bridge by 50%. They were built by the so-called "spinning-in-the-air" process, invented by John A. Roebling, but in many ways so much improved by Hildenbrand that he was enabled to build the cables of the Williamsburg Bridge in one-third the time of the older cables.

In 1904 Hildenbrand was appointed Consulting Engineer for the Westinghouse Electric & Manufacturing Co., of Pittsburg, to design the overhead steel structure for a single-phase electric system to be used in regular heavy railroad service, on the New York, New Haven & Hartford R. R. Mr. Hildenbrand held this position of

Consulting Engineer to the time of his death. In 1877 Mr. Hildenbrand published a treatise on the theory and construction of wire cables, which is still a standard work and the only book on this subject in existence. In 1888 he published a book on underground haulage of coal by wire rope which was awarded a medal and diploma of honor at the World's Fair at Chicago in 1893. In 1905 he published in book form, in the German language, one of his lectures on the history of suspension bridges from the earliest to the latest times. He also contributed many articles to the technical press and other publications, and many of his papers are contained in past volumes of Engineering News.

Mr. Hildenbrand became a member of the American Society of Civil Engineers in 1902. While his interest and enthusiasm was chiefly in his professional work, he had excellent taste in music. He married some eight years ago Miss Hubbard, daughter of Judge Hubbard, of Covington, Ky., and she survives him.

THE FAILURE OF THE CONCRETE ROOF OF THE LAWRENCE, MASS., FILTER.

On April 3, 1907, a large portion of the concrete roof of the addition to the sand filters at Lawrence, Mass., then under construction, collapsed while the forms were being removed and so seriously retarded the work on the increase in the water supply that $10,000 had to be expended the next summer in the purchase of water from neighboring communities. No authoritative statement of the causes of this accident had been made until Mr. Sanford E. Thompson, M. Am. Soc. C. E., read a paper at the meeting of the New England Water-Works Association in Boston, on Feb. 12, 1908, entitled "Investigation of Collapse of Filter Roof During Construction at Lawrence, Mass." This paper, which gives in detail the report made by Mr. Thompson to Mr. Morris Knowles, the designer of the plant, is abstracted below.

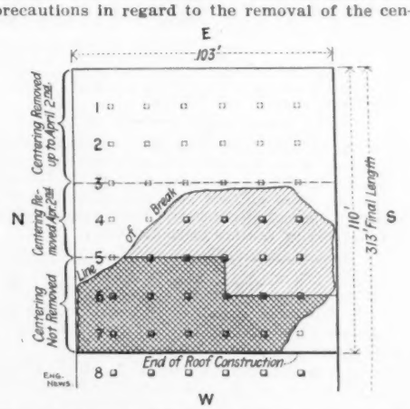
The addition to the Lawrence filters consists of a ¾-acre bed, 313 × 103 ft. in plan, built of concrete in 15-ft. square bays of groined arches, both in floor and on roof, which latter was supported from the floor by square piers. The roof arches span 15 ft. clear, are 6 ins. thick at crown and 22 ins. at the pier and are intended to carry a 3-ft. earth fill. All masonry is of non-reinforced concrete. In construction the concrete was laid in three divisions, first, the floor, with its groined arches; next, the columns, connected to the floor by two dowels of 1-in. iron pipe projecting about 8 ins. up into the columns, and, lastly, the roof of groined arches resting upon the columns and supported during construction upon the usual centering.

The accompanying figure illustrates the progress of the work up to the time of the accident. Construction was started early in 1906, but many delays retarded progress to such an extent that in November of that year only 8½ out of the 21 bays of the floor and 8 rows of piers were completed. At this time the roof was started at the east end and up to the closing down for the winter the work by weeks was as follows: Nov. 2, bays 0-2½; Nov. 9, bays 2½-3½; Nov. 23, bays 3½-5½; Dec. 17, bays 5½-7½. This last portion was ready for concrete early in December, but the temperature was so low that work was postponed until there was more favorable weather.

The specifications provided that any time after Nov. 13 the engineer could stop work in case he considered the temperature too low for successful concrete laying. All during November the average daily temperature varied between freezing and 50° F., with minimums below freezing. Beginning with December, freezing weather set in and lasted with few rises until the middle of March. The last concrete work was done on Dec. 17, but in all of the roof work precautions were taken to prevent the concrete from freezing. As soon as the concrete was laid in the roof bays, they were covered with straw to a depth of 6 ins., and later about 2½ ft. of sand and gravel was placed on top of this straw, this final dead load being placed in from two to three weeks after concreting.

During the winter the centering under the first three bays was removed, and on April 2, the day before the collapse, the removal of centering under the remaining bays was commenced, so that by night all falsework under the completed arches had been taken away except the half centers between the seventh and eighth rows of columns, the five centers of bay 7 and the four north arches of bay 6. On the morning of April 3, while the men were at work on this remaining portion, an ominous cracking was heard and, just after the last man had reached safety, the roof started to break about the middle of the fourth bay and the whole portion from that point to the piers last constructed collapsed. Subsequent examination showed that, though many of the piers were bent out of plumb and cracked considerably, only a few were turned over. In those that had failed a distinct line of cleavage between the column and the floor was noted.

In the opinion of Mr. Thompson, the roof as designed was amply strong; the causes that led to the failure lay in the construction. In this construction the composition of the concrete, the conditions of temperature under which it was deposited and the disregard of certain prescribed precautions in regard to the removal of the centering, all combined to precipitate a collapse, which probably would not have resulted from any one of these evidences of poor construction.



Plan of Concrete Filter Beds at Lawrence, Mass. (Showing stage of false work removal and location of roof collapse.)

An examination of the concrete immediately after the collapse showed that it was so soft and crumbly that it could be easily pulled to pieces. A month later it was somewhat harder, but still was not what would be called first-class concrete. In view of the extreme conditions of low temperature that prevailed during the laying of the roof and for a period of three months thereafter, it is quite probable that the strength of the concrete was materially affected by the low temperature. This is further proved by the increase in strength in the subsequent warmer period. It was noted that the straw protective coating contained considerable manure and an examination showed that where this manure occurred there was a discoloration of the surface of the concrete to the depth of about an inch. Although this discolored section was softer than the rest of the concrete, its area and depth were so small that it is probable it did not do more than contribute a small part to the weakness of the structure. However, it is additional proof of the well-evidenced fact that manure is a bad thing to place on green concrete.

In regard to the composition of the concrete, Mr. Thompson says:

> The sand and gravel used for the concrete were from a pile which had been excavated from the site of the filter. This material was screened into gravel for the coarse aggregate or ballast, and sand. Four samples of the sand were taken, and two samples of the screened gravel. Examination of the sand and of its analyses shows that it is all of medium quality, containing a somewhat excessive percentage of dirt for maximum strength, but none too much for water tightness. Examination of the analyses of the gravel shows a large per cent. of material under ¼-in., the sample with the least fine material screening 20% below ¼-in., although the contract specifies that "all particles smaller than ¼-in. shall be screened out." The specifications require proportions 1:3:5. If 20% of the gravel is below ¼-in., and therefore sand, the proportions are actually 1:4:4, thus giving a concrete greatly inferior in density and strength to that required by the specifications. Many of the gravel stones were covered with a coating of very fine clayey sand, which after drying in the laboratory and screening by hand for five minutes did not entirely come off. Tests by Mr. Howard A. Carson (recorded in the Seventh Annual Report of the Boston Transit Commission, 1901, p. 39) show one-third greater strength for concrete made from washed gravel than similar concrete made with gravel coated with a thin film of dirt. The excess of fine material tends to make the concrete not only weaker but slower in setting and hardening.
>
> The contract specifies that the gravel "shall be washed if necessary to render it sufficiently clean," but this was not done.

The specifications required the contractor to leave the centering under any bay of the roof until the five bays next succeeding had been placed. Evidently the contractor supposed that this provision was made to allow the concrete sufficient time to set, and when form removal began in the spring, as the concrete had then been in place some three months, he dismantled the centers regardless of the condition of the roof arches ahead. However, this requirement as to form removal was probably inserted to provide support for the arches under the progressive loading of the earth fill, which, unless the earth be stopped on a vertical plane over the piers or over the center of the arch, must cause a thrust down the arch ring and an overturning moment on the pier. This unbalanced loading of earth was actually made and undoubtedly distributed a thrust along the whole series of arches when the forming was removed. In addition, many diagonal braces were left in position which carried this thrust directly to the middle of piers which had nothing but their dead weight and the balanced thrust of the opposite arch to prevent them from overturning. In this connection Mr. Thompson's report reads partly as follows:

> The assumption that the thrust was a factor in the fall is strengthened by the facts that (1) one-half of the bay between column rows 3 and 4 was laid as late as November 23, thus having only 12 days more to set with the temperature slightly above freezing than the outer bays, and yet this bay suffered the removal of the forms without apparent damage; (2) the bay between column rows 5 and 6, half of which was laid on the final date, December 17, 1906, did not fall on the day when the forms were removed from the first portion of it, although unreinforced concrete falls quickly with but slight warning; (3) warning of failure was given by cracking of the centers and apparently commenced in about the middle of the reservoir near pier 8.3, which was tipped from the thrust of the diagonal braces against it.

Finally the report gives the following causes for the failure:

> (1) The thrust of the arches; (2) the weak condition of the concrete because of (a) low temperature which retarded the hardening; (b) improper screening of gravel producing an excess of sand which tends to make concrete slow in hardening; (c) dirty gravel, reducing the strength of the concrete.
>
> All of these sources were probably contributory to the fall. The subsequent hardening of the concrete indicates that no trouble could have ensued if the forms had remained for a longer time and until more arches had been placed.

THE ECONOMIZER attached to one of the boilers in the new power-house of the Hamilton Corporation, a cotton factory in Lowell, Mass., exploded on Feb. 19 and seriously injured several workmen who were installing the machinery.

A POWDER MILL EXPLOSION in the works of the General Explosive Co., at Wharton, N. J., on Feb. 18, caused the death of three workmen and wrecked the entire plant beyond repair, besides destroying considerable property in the vicinity. The packing house of the Hercules powder works at Pinole, Cal., blew up on Feb. 20 and killed four white men and 24 Chinamen—every man that was at work in the building.

A CAVE-IN in the Midvalley mine, near Mount Carmel, Pa., on Feb. 17, imprisoned 28 miners for about 12 hours. There was sufficient room and air in the confined space to allow the entombed men to work, and their

efforts, combined with the tunneling carried on from the outside, made a passageway out of the mine, through which all but one miner passed with safety. A falling timber killed this one man.

IN A GRADE-CROSSING DISASTER at Nyack, N. Y., on Feb. 23, six persons were killed and three injured. A carryall, containing a party of young people returning from a Washington's Birthday party, was crossing the tracks of the New York, Ontario & Western Ry., when the "Mountain Express," traveling at about 60 miles per hour, struck it. Testimony seems to show that the flagman at the crossing had the gates down to obstruct passage, but that the carryall was driven straight through the barrier.

BIDS FOR A DIRIGIBLE BALLOON for the Signal Corps, U. S. A., were opened at the Signal Office, War Department, Washington, D. C., on Feb. 15. The following bids were received:

Peter Cooper Hewitt, New York City, $20,000; 200 days.
H. B. Schiller, Philadelphia, $33,500; 180 days.
S. C. Rockman, Philadelphia, $25,000; 180 days.
Raymond Anglemire, Chicago, no price; 90 days.
Bumbaugh & Heimann, St. Louis, $10,000; 250 days.
Louis Lupetts, Paris, France, $20,000; 90 days.
Carl E. Miers, Frankfort, N. Y., $11,994; 120 days.
Wm. Reiferscheid & Co., Streator, Ill., $8,000; 150 days.
Chas. Ellis, Toledo, Ohio, $10,000; no time.
Thos. S. Baldwin, New York, $6,750; 150 days.

It will be remembered that bids were first advertised to be opened on Jan. 15 (see Engineering News, Jan. 23, p. 96). At this time six bids were received, but were rejected and new proposals were solicited (Engineering News, Feb. 6). It is reported that on Feb. 24 contract for one dirigible balloon was awarded to Mr. T. S. Baldwin, the lowest bidder.

THE CITY REPAIR DEPARTMENT of Chicago was described at some length in our issue of Dec. 5, 1907, and it was explained that all expenses of operating this division were covered by a 10% addition to the cost of the work as done for each of the several city departments. The annual report of Mr. W. D. Barber, Assistant Engineer of the Department of Public Works, who is in charge of this Division, shows that the operating expenses in 1907 were only about 6% of the cost of the labor and materials used. The report refers to the high degree of quality and economy represented by the work, due to the systematic records of labor and material for each individual piece of work.

PORTLAND CEMENT PRODUCTION in the United States during 1907 fell far short of the expected total, according to the advance report just issued by the U. S. Geological Survey. In our issue of Oct. 17, 1907, p. 406, we published an abstract of the last annual report of the Survey, in which the production curve for the past 25 years was given and the prediction made that the amount of Portland cement manufactured in 1907 would reach 60,000,000 bbls. However, returns from 87 plants, representing over 95% of the Portland cement production in the United States, indicate that in the calendar year 1907 only about 48,000,000 bbls. were produced in this country. If this be compared with the output of 46,463,424 bbls. in 1906, 35,246,812 bbls. in 1905, and 26,505,881 bbls. in 1904, it will be seen that the rate of increase has dropped materially. In connection with the report Mr. E. C. Eckel, who is in charge of the statistical work on cement for the Survey, says:

Though the production of 1907 shows an increase over that of 1906, it is slight as compared with past annual gains in the cement industry, indicating that the Portland cement production of the United States has now reached a stage in its development where it is directly and promptly affected by general business depression.

STATE MINE INSPECTORS have recently reported on the conditions in the Naomi mine, near Fayette, Pa., where a serious explosion occurred on Dec. 1. A special committee of mine inspectors was appointed by Jas. E. Roderick, Chief of the Pennsylvania Department of Mining, and its report and order to the Superintendent of the mine is printed in the Pittsburg "Dispatch," as follows:

The undersigned inspectors of bituminous coal mines have proceeded to said Naomi mine, where danger is alleged to exist, and after a full investigation of said mine we are agreed in the opinion that there is immediate danger therein arising from its gaseous condition, lack of proper ventilation, the use of open lamps therein and from deposits of coal dust which endanger the lives, health and security of the miners employed therein.

And you are hereby instructed to remove such conditions forthwith and provide at least 150,000 cu. ft. of air per minute at the inlet or inlets to the said mine, and have the same promptly and constantly circulated through all the working places, main entries and cross entries so as to dilute and carry off and render harmless the noxious and dangerous gases generated in said mine, and also that you adopt a more perfect system of ventilation and conduct the air by means of overcasts instead of the numerous doors which are now found in said mine.

To work said mine exclusively with locked safety lamps. To install and maintain a sufficient water system and to thoroughly water and keep wet the sides, roof and floor of all working places, main entries and cross entries and to thoroughly water, as herein described, all working places for a distance of not less than 100 ft. back from the working face immediately before any shot or shots are fired, otherwise we shall be obliged to apply to the Court of Common Pleas of the County of Fayette for an injunction to suspend the work in and about said mine in accordance with the provisions of Article XI. and Section 1 of the act of Assembly approved May 15, 1893, entitled "An act relating to bituminous coal mines and providing for the lives, health, safety and welfare of persons employed therein."

(Signed) HENRY LOUTTIT, First Bituminous District.
C. B. ROSS, Second Bituminous District.
I. G. ROBY, Fifth Bituminous District.
ELIAS PHILLIPS, Fourth Bituminous District.
JOSEPH KNAPPER, Eighth Bituminous District.
ALEXANDER McCANCH, Eleventh Bituminous District.

In addition to the dangers which you are instructed to remove from Naomi mine we would recommend to you the following additional precautions in order to protect the lives and health of the miners and mine itself:

First—That you allow no blasting on ends of pillars or pillar work.

Second—That you use the best flameless explosive.

Third—That competent shot-firers be selected and employed to prepare, charge and fire the shots after the workmen are out of the mine. The shot-firers to have the right to refuse to charge and fire any hole which, in their judgment, would be unsafe to fire, and all stemming to be done with clay or other incombustible material.

Fourth—That a different type of coal-cutting machine be substituted for the chain machines.

Fifth—Dust should not be allowed to accumulate in the mine.

Sixth—You should employ a sufficient number of fire bosses so that they can make a careful examination of the mine without undue haste.

Seventh—That you shall see to it that the mine foreman devotes his whole time to his duties in the mines as prescribed by the act of May 15, 1893.

Eighth—That the mine foreman shall maintain rigid discipline at all times.

Ninth—We also recommend that the system of mining shall be such that a greater percentage of the seam be removed from pillars so as to allow the overlying strata to fall, closing up all cavities where gas might accumulate and endanger life and property.

THE SAULT STE. MARIE CANAL, forming the western approach to the Poe lock, is to be widened to double its present width. At the draw span of the International Bridge there will now be a channel on each side of pivot pier instead of the present single channel which has to be used by vessels in both directions.

PERSONALS.

Mr. W. E. Condon, consulting civil engineer, has removed from Berkeley, Cal., to the French Bank Bldg., San Francisco.

Mr. Angus Sinclair, M. Am. Soc. M. E., editor of "Locomotive Engineering" is to be honored by Purdue University with the degree of Doctor of Engineering.

Mr. J. P. McCuen, Superintendent of Motive Power on the Queen & Crescent Route, has resigned, to take effect on March 1, the 26th anniversary of his connection with the road.

Mr. Russell B. Reid has severed his connection with the Fayerweather & Ladew Belt Mfg. concern and has become Eastern Sales Manager of the Republic Belting Co., of Cleveland, Ohio, with offices at 36 Vesey St., New York City.

Mr. Walter J. Francis, M. Am. Soc. C. E., lately the Assistant Manager of the Dominion Engineering & Construction Co., of Montreal, has opened offices as consulting engineer in the Sovereign Bank Building, Montreal, Canada.

Mr. Theodore Starrett, for many years partner in the Thompson-Starrett Co., engineers and contractors, has retired from that company and has opened offices at No. 2 Rector St., New York City, as adviser and consulting engineer in building construction and the management of construction operations.

Mr. W. M. Hobbs has resigned from the position of General Manager of the San Antonio & Arkansas Pass Ry. to become General Superintendent of the various Harriman lines in and about New Orleans, succeeding Mr. E. B. Cushing, who is to take charge of construction work between Lafayette and Baton Rouge.

Miss Nora Stanton Blatch, Jun. M. Am. Soc. C. E., and Dr. Lee De Forrest, M. Am. Inst. E. E., were married in New York City on Feb. 25, 1908. Mrs. De Forrest was graduated from Cornell University in 1905 with the degree of Civil Engineer and has since been engaged in engineering work with the Board of Water Supply of New York City. Dr. De Forrest is best known as the inventor of systems of wireless telegraphy and wireless telephony.

Prof. Henry Taylor Bovey, Dean of the Faculty of Applied Science at McGill University, Toronto, Ont., is to return to England to become Rector of the Imperial College of Science and Technology, a new engineering university now in process of organization in London. Dean Bovey is a graduate of Cambridge University in England, and for the past 30 years has been in active service in the engineering school at McGill. He is the author of several well-known text-books.

Mr. Harold U. Wallace, M. Am. Soc. C. E., has resigned from the presidency of the Wallace-Coates Engineering Co., of Chicago, to become General Manager of the Chicago, Lake Shore & South Bend Ry. Co., a new single-phase electrical line. Up to the time he became connected with the Wallace-Coates Co., Mr. Wallace was in the engineering and transportation departments of the Illinois Central Ry., serving as Chief Engineer for three years. He has also been Third Vice-President of the J. G. White Co.

Obituary.

Lewis Hazelton Taylor, honorary President of the Taylor Iron & Steel Co., of High Bridge, N. J., died at his home at High Bridge on Feb. 18 at the advanced age of 97 years.

Wilhelm Hildenbrand, M. Am. Soc. C. E., the distinguished bridge engineer, died at his home in New York City on Feb. 21. An extended biography of Mr. Hildenbrand will be found on another page of this issue.

W. F. Hallstead, for many years General Manager of the Delaware, Lackawanna & Western R. R., died on Feb. 23, aged 71 years. Mr. Hallstead entered the service of the Lackawanna in 1852 and served in all capacities from brakeman to general manager, which last position he held from 1886 until his retirement in 1899.

James Dun, M. Am. Soc. C. E., Chief Engineer of the Atchison, Topeka & Santa Fe Ry. system, died at St. Augustine, Fla., on Feb. 23, at the age of 53 years. Mr. Dun entered railway service in 1866 as chainman on the Indianapolis & Cincinnati R. R. and became Chief Engineer of the Santa Fe in 1890, serving continuously in that capacity until his death.

ENGINEERING SOCIETIES.

COMING MEETINGS.

AMERICAN RAILWAY ENGINEERING AND MAINTENANCE OF WAY ASSOCIATION.
March 17-19, 1908. Annual convention at Chicago, Ill. Secy., L. C. Fritch, 1562 Monadnock Block, Chicago.

AMERICAN RAILWAY ASSOCIATION.
April 22, 1908. Semi-annual session at New York City. Secy., W. F. Allen, 24 Park Place, New York.

AMERICAN ELECTROCHEMICAL SOCIETY.
May 1-2, 1908. Annual meeting at Albany, N. Y. Secy., Joseph W. Richards, Lehigh University, South Bethlehem, Pa.

AMERICAN WATER-WORKS ASSOCIATION.
May 11-16, 1908. Annual meeting at Washington, D. C. Secy., John M. Diven, 14 George St., Charleston, S. C.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
June 23-26, 1908. Annual convention at Denver, Colo. Secy., C. W. Hunt, 220 West 57th St., New York City.

AMERICAN SOCIETY OF MECHANICAL ENGINEERS.
June 23-26, 1908. Annual convention at Detroit, Mich. Secy., C. W. Rice, 29 West 39th St., New York City.

THE MUNICIPAL ENGINEERS OF THE CITY OF NEW YORK held their regular meeting on Feb. 26. The paper of the evening was by Mr. W. B. Fuller, upon "The Proposed Filtration of the Croton Water Supply of New York City."

ENGINEERS' CLUB OF BALTIMORE.—At the next meeting of the club, March 7, 1908, Mr. Swepson Earle, Engineer of the Maryland Shellfish Commission, will read a paper on "The Maryland Oyster Survey and Oyster Farming in the Chesapeake Bay."

THE APPALACHIAN ENGINEERING ASSOCIATION has elected the following officers for the ensuing year: President, Mr. A. Wagner; Vice-President, Mr. Ray V. Hennen; Treasurer, Mr. R. H. Buchanan, and Secretary, Dr. Henry M. Payne, Morgantown, W. Va.

AMERICAN SOCIETY OF MECHANICAL ENGINEERS.—The regular March meeting of the society will be held at the Engineering Societies' Building, 29 West 39th St., New York City, on March 10. The paper of the evening will be by Dr. Charles P. Steinmetz, Past President of the American Institute of Electrical Engineering, on "The Steam Path of the Steam Turbine."

AMERICAN INSTITUTE OF ELECTRICAL ENGINEERS.—A special meeting will be held in the auditorium of the Engineering Societies' Building, 29 West 39th St., New York City, on Thursday evening, March 5, 1908, at eight o'clock, at which Mr. Gifford Pinchot, Forester of the United States, will deliver an illustrated lecture on Forest Preservation. This lecture will be open to the public and tickets will not be required.

THE RAILWAY SIGNAL ASSOCIATION.—The Committee on Automatic Stops and Cab Signals has been instructed to report especially on: (1) Various cab signals and automatic stops which are worthy of recommendation. (2) On proper location of automatic stop trips for varied conditions of traffic. (3) The advisability of the use of cab signals as a substitute for fixed signals or adjuncts thereto.

In order that the committee may become informed regarding the latest state of the art and obtain the advice and assistance of those interested, inventors and manufacturers of devices of the kind referred to are requested to communicate with the chairman either in person or by letter, and furnish full description, patent specifications if possible, and drawings showing their devices.

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